City of Albuquerque - Public Works Department/Hydrology Division

SO. EUBANK STORM DRAINAGE ANALYSIS

AMMENDMENT #2

NOVEMBER 2, 1998

SMITH ENGINEERING, INC.

Smith Engineering Company

A Full Service Engineering Company

November 2, 1998

Loren D. Meinz
Civil Engineer
City of Albuquerque Public Works Dept
Hydrology Division
P.O. Box 1293
Albuquerque, NM 87103

Re: Amendment #2 for the South Eubank Area Storm Drainage Analysis (November 1995)

Dear Mr. Meinz:

Smith Engineering Company (SEC) is pleased to submit to your office an amendment to the hydrology for the South Eubank Area Storm Drainage Analysis (November 1995). This Amendment #2 updates the hydrology that contributes to the proposed Eubank Storm Drain between Opportunity Boulevard (Gibson) and the Tijeras Arroyo. A total drainage area of about 235 acres has been analyzed to determine the allowable discharge rates for the 100-year 24-hour storm from undeveloped properties. A map of the study area is found on Plate 1E.

Hydrologic Assumptions

SEC has delineated eight sub-basins that may contribute runoff to the proposed South Eubank Storm Drain (to be located adjacent to Eubank Boulevard). Three of the eight sub-basins (605R, 606AR, and 608R) are fully developed and currently discharge to Eubank Boulevard. These sub-basins are designed to free discharge directly to the new South Eubank Storm Drain at stations 66+23, 61+59, and 42+23, respectively.

The remaining five sub-basins are either undeveloped (606R, 607AR, 607R, and 609R) or developed and not currently draining to Eubank Boulevard (604R). Sub-basins 604R is assumed to enter and join flows from sub-basin 606R. The combined flow is attenuated through some form of detention storage and then flows south in storm drain pipe "A" to sub-basin 607R. Peak flows from sub-basin 607R are attenuated through some sort of detention storage and then this flow joins flow from 606R and is conveyed west to Eubank Boulevard through storm drain pipe "B". Flow from sub-basin 607AR is attenuated through some form of detention storage and then it combines with flow from pipe "B" . This total flow then discharges to the South Eubank Storm Drain at Station 50+68. Flow from sub-basin 609R is attenuated through some form of detention storage and the reduced peak flow is discharged to the South Eubank Storm Drain at Station 33+00.

Changes to Option B

All hydrologic analysis was performed for future developed conditions with Option B in place. The following changes were made to AHYMO Option B model used in the "South Eubank Area Storm Drainage Analysis Report":

- Sub-basins 605R, 606AR, 606R, 604R, 607AR, 607R, 608R, and 609R have been newly delineated and the drainage areas have been recalculated based on more accurate property boundary maps.
- Detention storage has been added to sub-basins 606R, 604R, 607R, 607AR, and 609R in order to limit the allowable discharge to 1.3 cfs/acre.
- Storm drain pipes "A" and "B" have been added to the AHYMO model.
- Sub-basins 401, 601, and 603, located upstream of study area, have also been added to the model.

6400 Uptown Boulevard, N.E. Suite 500E Albuquerque, N. M. 87110 Telephone 505/884-0700 Fax 505/884-2376

- The land treatment percentages for sub-basin 608R have been adjusted to 70% "A" and 30% "C" for this fully developed PNM site.
- The Manzano Mesa Detention Basin's stage-storage-outflow table has been updated.

Hydrologic Analysis

The AHYMO rainfall-runoff model, SEHYDAM2.DAT, was used to compute the 100-year 24-hour peak flows at stations along the proposed South Eubank Storm Drain. A summary of the basin parameters is shown in Table 1. A summary of the peak flows at the analysis points is shown in Table 2. A summary table from the AHYMO model is also attached as Appendix A.

The allowable discharge from sub-basins 606R, 604R, 607AR, 607R, and 609R is limited to 1.3 cfs/acre in order to prevent the overtaxing of the proposed South Eubank Storm Drain design. Developed properties 605R, 606AR, and 608R are permitted to free discharge to the new South Eubank Storm Drain.

The storm drain pipes "A" and "B" are estimated to be 42" and 48" in diameter based on Manning's equation for full pipe flow. These pipe sizes are estimates only. We have executed the routing of flows from sub-basins 604R, 606R, and 607R as discussed in previous paragraphs. Alternative routings are possible, but the hydraulic grade line in the proposed South Eubank Storm Drain must be reanalyzed.

Hydraulic Analysis

The computed peak flows from the AHYMO model were input into the program, StormCAD (v 1.5), to compute the hydraulic grade line for the proposed South Eubank Storm Drain. The results are shown in Appendix B.

Recommendations

- Allow sub-basins 605R, 606AR, and 608R to free discharge to the new South Eubank Storm Drain.
- Restrict the allowable discharge from sub-basins 604R, 606R, 607R, 607AR, and 609R to 1.3 cfs/acre.
- If alternative routings are proposed, other than used in this amendment, then a new hydraulic analysis should be performed to ensure that the hydraulic grade line in the South Eubank Storm Drain is not adversely impacted.

Should you have any questions or comments about this analysis please call Steve Kemna at (505) 884-0700.

Sincerely,

Smith Engineering Company

Stephen Paul Kemna

Stephen P. Kemna, P.E.

6400 Uptown Boulevard, N.E. Suite 500E Albuquerque, N. M. 87110 Telephone 505/884-0700 Fax 505/884-2376

TABLE 1. SUMMARY OF BASIN PARAMETERS, RUNOFF VOLUME AND PEAK FLOWS

Basin ID	Area (~~ac~~) Sq. Mi.	%A	%B	%C	%D	Tp (hrs)	V100 (ac-ft)	Q100 (cfs)	Q100 Allowable (at 1.3 cfs/acre)
605R	0.02906	0	15	0	85	0.14	3.9	82	N/A
606AR	0.02906	0	31	0	69	0.17	3.4	68	N/A
606R	0.05453	0	31	0	69	0.32	6.4	89	45
604R	0.06718	10	20	10	60	0.23	7.3	125	56
607AR	0.03031	0	30	0	70	0.14	3.6	78	25
608R	0.04	70	5	30	0	0.23	1.6	37	N/A
607R	0.06359	0	30	0	70	0.26	7.6	119	53
609R	0.05375	0	50	0	50	0.48	5.3	57	45

TABLE 2. PEAK FLOW AT ANALYSIS POINTS

Analysis Point	100-Year 24 Hour Peak Flow (cfs)
70+92	438
66+23	490
61+59	554
50+68	627
42+23	663
33+00	673

SUPPLEMENT TO AMMENDMENT #2
DETENTION STORAGE ASSUMED IN AHYMO HYDROLOGY MODEL (sehydam2.dat)

Basin ID	100-Year 24-Hour Peak Storage Volume (acre-feet)
607AR	1.4
606R	4.4
607R	2.6
609R	1.1

APPENDIX "A"

```
AHYMO SUMMARY TABLE (AHYMO194) - AMAFCA Hydrologic Model - January, 1994          RUN DATE (MON/DAY/YR) =10/30/1998
INPUT FILE = SEHYDAM2.DAT                                                                  USER NO.= T_DALTON.S94

                              FROM  TO                    PEAK        RUNOFF                TIME TO    CFS      PAGE =   1
                  HYDROGRAPH   ID   ID        AREA     DISCHARGE     VOLUME      RUNOFF      PEAK     PER
COMMAND        IDENTIFICATION  NO.  NO.     (SQ MI)      (CFS)       (AC-FT)    (INCHES)   (HOURS)   ACRE      NOTATION

*S  MODIFIED OCTOBER, 1998 FOR AMMENDMENT #2 TO SOUTH EUB DR. REPORT
*S  REVISION 3: FILE NAME SEHYDAM2.DAT
START                                                                                                  TIME=       .00
RAINFALL  TYPE= 2                                                                                      RAIN24=   3.170
*S  24 HR ;  100-YEAR STORM
*S  ONE-HOUR=2.07
*S  SIX-HOUR=2.56
*S  24 -HOUR=3.17
*S
*S START SOUTH EUBANK AHYMO MODEL - OPTION B
*S
*S      INCLUDES PROPOSED 96" PIPE AND DETENTION BASIN WITH 66" OUTLET PIPE
*S      THAT PICK UP FLOWS FROM BASIN 501, 502, 503, 504, 401, 403
*S      DOES PICK UP PIPE FLOW FROM CENTRAL AND ELIZABETH
*S      INCREASE PIPE SIZE IN JANE STREET AND DIVERTS THIS FLOW NORTH WARD
*S      TO I-40 DIVERSION CHANNEL
*S      TAKE FLOW FROM 54" STUB-OUT AT CENTRAL AND EUBANK TO EUBANK LINE
*S
*S HYD 101 IS OLF THAT IS AT THE BEGINNING OF THE CENTRAL SD SYSTEM
COMPUTE NM HYD        101.00   -    11        .03319       103.12       4.982      2.81475    1.500    4.855 PER IMP= 100.00
*S ROUTING 101 THRU 102 ON CENTRAL AVENUE'S NORTH SIDE
ROUTE                 101.02   11   12        .03319        91.32       4.972      2.80909    1.533    4.299
COMPUTE NM HYD        102.00   -    11        .05253       149.46       7.217      2.57618    1.500    4.446 PER IMP=  88.00
ADD HYD               102.10 11&12  13        .08572       237.16      12.190      2.66636    1.533    4.323
*S DIVIDE BETWEEN MH# S-132 TO MH# S-262
*S****     PIPE HYD CB1 AND OLF HYD 102.13 IS AP #9     *****
DIVIDE HYD               CB1   13   11        .05737        61.00       8.158      2.66636    1.367    1.661
                      102.13  AND   12        .02835       176.16       4.032      2.66636    1.533    9.708
*S OLF HYD 102.13 THRU 103 ON CENTRAL STREET'S NORTH SIDE
ROUTE                 102.12   12   14        .02835       137.98       4.032      2.66654    1.600    7.604
COMPUTE NM HYD        103.00   -    13        .02347        49.09       2.934      2.34361    1.600    3.268 PER IMP=  77.00
ADD HYD               103.13 13&14  12        .05182       187.07       6.966      2.52027    1.600    5.640
*S DIVIDE CATCH BASINS AND CATTLE GUARD BETWEEN MH# S-102  TO  MH# S-132
*S OLF HYD 103.03 IS AT JUAN TABO AND CENTRAL ON THE NORTH SIDE
*S****     OLF HYD 103.03 IS AP #13     *****
DIVIDE HYD               CB2   12   14        .04177        99.00       5.614      2.52027    1.500    3.703
                      103.03  AND   15        .01005        88.07       1.351      2.52027    1.600   13.688
COMPUTE NM HYD        201.00   -    21        .02777        84.21       4.025      2.71786    1.500    4.738 PER IMP=  95.00
*S EXISTING DETENTION BASIN IN BASIN 201
*S ROUTING RESERVOIR WITH SMALL ORIFICE PIPES AND 20' WEIR
ROUTE RESERVOIR      201.RES   21   22        .02777        23.79       3.930      2.65342    1.933    1.339 AC-FT=    2.055
*S
*S       OUTFLOW(CFS)    STORAGE(AC-FT)    ELEV(FT)
*S           0                0              5620
*S           2.5              0.468          5621
*S           3.8              0.937          5622
*S           4.8              1.405          5623
*S           5.6              1.873          5624
*S          29.0              2.107          5624.5
*S          70.0              2.342          5625
*S
*S ROUTING 201.RES THRU 202
ROUTE                 201.02   22   21        .02777        20.59       3.912      2.64132    2.066    1.159
COMPUTE NM HYD        202.00   -    22        .04041       103.90       5.841      2.71003    1.567    4.017 PER IMP=  95.00
COMPUTE NM HYD        209.00   -    25        .05761       119.24       6.353      2.06781    1.567    3.234 PER IMP=  62.00
ADD HYD               202.10 21&22  24        .06818       105.99       9.753      2.68204    1.567    2.429
ADD HYD               209.10 24&25  23        .12579       225.22      16.106      2.40073    1.567    2.798
*S DIVIDE OLF HYD 209.1 BY PROPOSED TOMLINSON PIPE NETWORK
*S****     OLF HYD 209.13 IS AP #1     *****
DIVIDE HYD          DIVTOM.1   23   71        .11369       125.00      14.556      2.40073    1.467    1.718
                      209.13  AND   24        .01210       100.22       1.550      2.40073    1.567   12.938
*S ROUTE DIVTOM.1 THRU 210 IN 48" AND 54" PIPE
*S Q(CAP) = 208 CFS, AT SLOPE=0.013
ROUTE               DIVTOM.12   71   72        .11369       125.00      14.535      2.39718    1.766    1.718
*S ROUTING 209.13 THRU 210 ON TOMLINSON DRIVE TO OUTFALL
ROUTE                 207C.12   24   22        .01210        65.76       1.550      2.40116    1.633    8.488
COMPUTE NM HYD        203.00   -    21        .08386       172.02      10.382      2.32120    1.600    3.205 PER IMP=  75.00
*S DIVIDE SOUTH SIDE CENTRAL AVENUE BETWEEN MH# S-132  AND  MH# S-262(CHECK TO S
*S****     OLF HYD 203.03 IS AP #10     *****
```

```
                                FROM  TO                    PEAK        RUNOFF                TIME TO    CFS
                   HYDROGRAPH   ID    ID        AREA     DISCHARGE     VOLUME      RUNOFF      PEAK     PER
COMMAND          IDENTIFICATION NO.   NO.      (SQ MI)     (CFS)       (AC-FT)    (INCHES)   (HOURS)   ACRE      NOTATION

DIVIDE HYD                CB5   21    24       .01273       1.00        1.576     2.32120     .967     .123
                       203.03  AND    25       .07113     171.02        8.805     2.32120    1.600    3.757
*S****     PIPE HYD CB5.1 IS AP #10     *****
ADD HYD                 CB5.1 24&11   21       .07010      62.00        9.734     2.60366    1.367    1.382
*S FROM MH# S-102  TO  MH# S-132
*S Q(CAP) = 73  CFS
ROUTE                  CB5.12   21    27       .07010      62.00        9.710     2.59718    1.966    1.382
*S ROUTING 203.03 THRU 204 ON CENTRAL AVENUE'S SOUTH SIDE
ROUTE                  203.02   25    23       .07113     152.20        8.801     2.31999    1.667    3.344
COMPUTE NM HYD         204.00    -    21       .06581     141.97        8.903     2.53659    1.600    3.371 PER IMP=  86.00
ADD HYD                204.10 23&21   24       .13694     287.70       17.704     2.42408    1.633    3.283
*S DIVIDE SOUTH SIDE CENTRAL AVENUE BETWEEN MH# S-102  AND  MH# S-123
*S OLF HYD 204.13 IS AT JUAN TABO AND CENTRAL ON THE SOUTH SIDE
*S****     OLF HYD 204.13 IS AP #14     *****
DIVIDE HYD                CB6   24    25       .10223     117.00       13.217     2.42408    1.467    1.788
                       204.13  AND    86       .03470     170.70        4.486     2.42408    1.633    7.686
ADD HYD                 CB6.1 27&25   21       .17234     179.00       22.927     2.49449    1.966    1.623
*S TOTAL PIPE FLOW AT JUAN TABO AND CENTRAL
*S Q(CAP) AT MH# S-102 IS 260 CFS
*S****     PIPE HYD CB6.2 IS AP #13 AND AP #14     *****
ADD HYD                 CB6.2 14&21   99       .21411     278.00       28.542     2.49952    1.766    2.029
*S BASIN 400 IS SOUTH OF CENTRAL
COMPUTE NM HYD         401.00    -    41       .03371      73.42        4.734     2.63296    1.633    3.403 PER IMP=  90.00
COMPUTE NM HYD         402.00    -    42       .03516      76.91        5.238     2.79322    1.633    3.418 PER IMP=  99.00
*S OLF HYD 103.13 THRU 303 AND 304 ON NORTH SIDE OF CENTRAL STREET
ROUTE                  103.12   15    43       .01005      49.25        1.351     2.52078    1.667    7.654
*S OLF HYD 204.13 THRU 402 ON SOUTH SIDE CENTRAL STREET
ROUTE                  204.12   86    44       .03470     115.32        4.487     2.42423    1.766    5.192
ADD HYD                402.10 42&44   45       .06986     181.32        9.725     2.60991    1.733    4.055
*S DIVIDE BY CB ON NORTH SIDE CENTRAL STREET BETWEEN MH# S-42  AND MH# S-102
*S OLF HYD 103.13 IS FLOW ON CENTRAL'S NORTH SIDE AT ELIZABETH AND CENTRAL
*S****     OLF HYD 103.13 IS AP #15     *****
DIVIDE HYD                CB14   43    42       .00975      42.00        1.310     2.52066    1.633    6.732
                       103.13  AND    44       .00031       7.25         .041     2.52066    1.667   37.118
*S DIVIDE BY CB ON SOUTH SIDE CENTRAL STREET BETWEEN MH# S-42  AND MH# S-102
*S OLF HYD 402.13 IS CENTRAL'S TOTAL SOUTH SIDE FLOW AT ELIZABETH AND CENTRAL
*S****     OLF HYD 402.13 IS AP #16     *****
DIVIDE HYD                CB15   45    46       .04141      49.00        5.764     2.60991    1.500    1.849
                       402.13  AND    47       .02845     132.32        3.961     2.60991    1.733    7.266
*S FROM MH# S-42  TO  MH# S-102
*S Q(CAP) = 276 CFS
ROUTE                  CB6.22   99    43       .21411     277.94       28.484     2.49447    1.766    2.028
ADD HYD                CB14.1 42&43   45       .22385     319.72       29.795     2.49561    1.700    2.232
*S TOTAL PIPE FLOW AT CENTRAL AND ELIZABETH
*S****     PIPE HYD CB15.1 IS AP #15 AND AP #16     *****
ADD HYD                CB15.1 45&46   42       .26526     368.72       35.559     2.51346    1.700    2.172
*S ROUTE PIPE HYD "CB15.1" THRU 403 IN 84" PIPE ON ELIZABETH ST TO COCHITI RD
*S 84" DIAM; 380 FT ; S=0.004
ROUTE                 CB15.12   42     2       .26526     368.88       35.548     2.51272    1.667    2.173
*S TOTAL PIPE FLOW AT ELIZABETH AND COCHITI
ADD HYD               401.1CB  2&41     3       .29897     439.79       40.282     2.52628    1.667    2.298
*S 84" DIAM; 720 FT; S=0.003; Q(CAP)=500 CFS
ROUTE                 P401.CB    3     2       .29897     439.20       40.254     2.52452    1.667    2.295
COMPUTE NM HYD         SEA403    -     1       .05744     110.01        6.746     2.20216    1.600    2.992 PER IMP=  70.00
ADD HYD                P403.1  1& 2    3       .35641     543.18       47.000     2.47257    1.667    2.381
*S 96" DIAM; 200 FT; S=0.003; Q(CAP)=500 CFS
ROUTE                 P403.12    3     2       .35641     543.68       46.991     2.47211    1.667    2.383
COMPUTE NM HYD         SEA501    -     1       .02795      38.63        3.885     2.60590    1.933    2.159 PER IMP=  90.00
ADD HYD                P102.1  1& 2    3       .38436     565.11       50.876     2.48184    1.667    2.297
*S 108" DIAM; 480 FT; S=0.0051
ROUTE                 P102.12    3    90       .38436     564.21       50.858     2.48098    1.667    2.294
*S
*S HYDROLOGY PROGRAM FROM AVID ENG; APRIL 14, 1997
*S
*S ADD BASINS 5 AND 3 TOGETHER AND ROUTE TO STORM SEWER AT INLET 13.10
COMPUTE NM HYD           5.00    -     1       .00567      13.37         .558     1.84577    1.500    3.685 PER IMP=  50.00
COMPUTE NM HYD           3.00    -     2       .00711      16.76         .700     1.84577    1.500    3.684 PER IMP=  50.00
ADD HYD                  5.00  1& 2    3       .01278      30.13        1.258     1.84577    1.500    3.684
*S STORM SEWER FLOW AT INLET 13.10
ROUTE                    3.10    3    43       .01278      30.14        1.258     1.84570    1.500    3.685
*S ADD BASINS 13 AND 15 TOGETHER AND ROUTE TO STORM SEWER AT INLET 13.20
COMPUTE NM HYD          15.00    -     1       .00650      15.33         .640     1.84577    1.500    3.684 PER IMP=  50.00
```

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
COMPUTE NM HYD	13.00	-	2	.00487	11.49	.479	1.84578	1.500	3.686	PER IMP= 50.00
ADD HYD	13.10	1& 2	3	.01137	26.81	1.119	1.84577	1.500	3.685	
*S STORM SEWER FLOW AT INLET 13.20										
ROUTE	13.10	3	44	.01137	26.80	1.119	1.84572	1.500	3.683	
*S ADD PIPE FLOWS AT AP-13 AND ROUTE OUT TO CITY DRAINAGE										
ADD HYD	13.F	44&43	44	.02415	56.94	2.377	1.84571	1.500	3.684	
*S TOTAL STORM SEWER FLOW ENTERING CITY STORM SEWER AT AP-1										
ROUTE	13.F	44	45	.02415	56.86	2.377	1.84552	1.500	3.679	
ADD HYD	13.F1	45&90	3	.40851	594.79	53.235	2.44341	1.667	2.275	
*S 108" DIAM; 650 FT; S=0.0051										
ROUTE	13.F12	3	90	.40851	593.49	53.211	2.44228	1.667	2.270	
*S ADD BASINS 11 AND 9 TOGETHER AND ROUTE TO STORM SEWER AT AP-9										
COMPUTE NM HYD	11.00	-	1	.01177	27.74	1.159	1.84577	1.500	3.682	PER IMP= 50.00
COMPUTE NM HYD	9.00	-	2	.00706	16.65	.695	1.84577	1.500	3.684	PER IMP= 50.00
ADD HYD	45.20	1& 2	3	.01883	44.38	1.854	1.84577	1.500	3.683	
*S STORM SEWER FLOW AT AP-9										
ROUTE	9.10	3	41	.01883	43.28	1.852	1.84432	1.533	3.591	
*S ADD BASINS 7 AND 27 TOGETHER AND ROUTE TO AP-45 VIA SURFACE FLOW										
COMPUTE NM HYD	7.00	-	1	.00647	15.26	.637	1.84577	1.500	3.684	PER IMP= 50.00
COMPUTE NM HYD	27.00	-	2	.00316	7.46	.311	1.84579	1.500	3.689	PER IMP= 50.00
*S SURFACE FLOW AT AP-45 DUE TO BASIN 7 AND 27										
ADD HYD	27.00	1& 2	3	.00963	22.72	.948	1.84578	1.500	3.686	
*S ADD BASINS 35 AND 33 TOGETHER AND ROUTE TO STORM SEWER AT AP-33										
COMPUTE NM HYD	35.00	-	1	.00838	19.75	.825	1.84577	1.500	3.683	PER IMP= 50.00
COMPUTE NM HYD	33.00	-	2	.00873	20.58	.859	1.84577	1.500	3.683	PER IMP= 50.00
ADD HYD	35.00	1& 2	4	.01711	40.33	1.684	1.84577	1.500	3.683	
*S STORM SEWER FLOW AT AP-33										
ROUTE	29.10	4	42	.01711	39.98	1.684	1.84530	1.500	3.651	
COMPUTE NM HYD	29.00	-	1	.00817	19.26	.804	1.84577	1.500	3.683	PER IMP= 50.00
*S ADD BASINS 29 AND 45 AND ROUTE TO AP-45 VIA SURFACE FLOW										
COMPUTE NM HYD	45.00	-	2	.00417	9.84	.411	1.84578	1.500	3.687	PER IMP= 50.00
ADD HYD	45.10	1& 2	4	.01234	29.10	1.215	1.84577	1.500	3.685	
*S TOTAL SURFACE FLOW AT AP-45										
ADD HYD	45.10	3& 4	6	.02197	51.81	2.163	1.84578	1.500	3.685	
*S ADD BASINS 51 AND 57 AND ROUTE TO AP-29 VIA STORM SEWER										
COMPUTE NM HYD	51.00	-	1	.00814	19.19	.801	1.84577	1.500	3.683	PER IMP= 50.00
COMPUTE NM HYD	57.00	-	2	.01178	27.76	1.160	1.84577	1.500	3.682	PER IMP= 50.00
ADD HYD	57.00	1& 2	5	.01992	46.95	1.961	1.84577	1.500	3.683	
ROUTE	42.00	5	43	.01992	46.06	1.960	1.84460	1.533	3.613	
ADD HYD	29.10	42&43	43	.03703	85.66	3.644	1.84493	1.533	3.614	
ROUTE	45.30	43	42	.03703	86.26	3.642	1.84437	1.533	3.640	
*S ADD STORM SEWER FLOWS FROM BASINS 9, 11, 35, 33, 51 AND 57										
ADD HYD	45.40	41&42	41	.05586	129.54	5.495	1.84435	1.533	3.623	
*S ADD SUFRACE FLOWS TO STORM SEWER FLOWS AT AP-45										
ADD HYD	45.F	41& 6	41	.07783	179.46	7.657	1.84475	1.533	3.603	
*S TOTAL STORM SEWER FLOWS AT AP-45										
ROUTE	45.F	41	40	.07783	179.78	7.654	1.84387	1.533	3.609	
*S SURFACE FLOW FROM BASIN 25										
COMPUTE NM HYD	25.00	-	3	.00998	23.52	.982	1.84577	1.500	3.683	PER IMP= 50.00
*S SURFACE FLOW FROM BASIN 23 TO INLET 33										
COMPUTE NM HYD	23.00	-	1	.00552	13.02	.543	1.84577	1.500	3.685	PER IMP= 50.00
*S STORM SEWER FLOW FROM INLET 33 TO MAIN STORM SEWER AT AP-33										
ROUTE	23.10	1	41	.00552	13.04	.543	1.84572	1.500	3.691	
ADD HYD	23.F	40&41	42	.08335	192.38	8.197	1.84400	1.533	3.606	
*S MAIN STORM SEWER FLOW AT AP-33										
ROUTE	23.F	42	40	.08335	192.10	8.196	1.84367	1.533	3.601	
*S SURFACE FLOW FROM BASIN 21 TO INLET 21										
COMPUTE NM HYD	21.00	-	1	.00572	13.49	.563	1.84577	1.500	3.685	PER IMP= 50.00
*S STORM SEWER FLOW FROM BASIN 21 TO MAIN STORM SEWER AT AP-21										
ROUTE	21.10	1	41	.00572	13.51	.563	1.84573	1.500	3.691	
ADD HYD	21.F	40&41	42	.08907	205.14	8.759	1.84380	1.533	3.599	
*S MAIN STORM SEWER FLOW AT AP-21										
ROUTE	21.F	42	40	.08907	204.89	8.758	1.84354	1.533	3.594	
*S SURFACE FLOW FROM BASIN 19										
COMPUTE NM HYD	19.00	-	4	.00528	12.45	.520	1.84578	1.500	3.685	PER IMP= 50.00
ADD HYD	39.10	3& 4	3	.01526	35.98	1.502	1.84577	1.500	3.684	
*S SURFACE FLOW FROM BASIN 39										
COMPUTE NM HYD	39.00	-	5	.00517	12.19	.509	1.84578	1.500	3.685	PER IMP= 50.00
*S TOTAL SURFACE FLOW AT AP-39 DUE TO BASINS 25, 19 AND 39										
ADD HYD	39.20	3& 5	3	.02043	48.17	2.011	1.84577	1.500	3.684	
COMPUTE NM HYD	43.00	-	1	.00995	23.45	.979	1.84577	1.500	3.683	PER IMP= 50.00
COMPUTE NM HYD	41.00	-	2	.00569	13.42	.560	1.84577	1.500	3.685	PER IMP= 50.00
*S SURFACE FLOW FROM BASINS 41 AND 43 TO INLET 39										

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
ADD HYD	43.00	1& 2	4	.01564	36.87	1.540	1.84577	1.500	3.684	
*S STORM SEWER FLOW FROM INLET 39 TO AP-39										
ROUTE	39.10	4	42	.01564	36.64	1.539	1.84543	1.500	3.660	
*S STORM SEWER FLOW FROM INLET 39 ADD TO MAIN STORM SEWER AT AP-39										
ADD HYD	39.20	40&42	40	.10471	241.04	10.297	1.84382	1.533	3.597	
*S SURFACE FLOWS ADD TO STORM SEWER AT AP-39										
ADD HYD	39.F	40& 3	42	.12514	287.46	12.308	1.84414	1.533	3.589	
*S TOTAL STORM SEWER FLOW AT AP-39										
ROUTE	39.F	42	40	.12514	287.58	12.307	1.84397	1.533	3.591	
*S ADD BASINS 49, 55, 53 AND 37 TOGETHER AND ROUTE TO AP-37										
COMPUTE NM HYD	49.00	-	1	.00788	20.01	.853	2.02978	1.500	3.968	PER IMP= 57.14
COMPUTE NM HYD	55.00	-	2	.00598	14.10	.589	1.84578	1.500	3.685	PER IMP= 50.00
ADD HYD	49.00	1& 2	3	.01386	34.11	1.442	1.95039	1.500	3.846	
COMPUTE NM HYD	53.00	-	1	.00238	6.07	.262	2.06613	1.500	3.983	PER IMP= 60.50
ADD HYD	49.00	1& 3	3	.01624	40.18	1.704	1.96735	1.500	3.866	
COMPUTE NM HYD	37.00	-	1	.00264	7.88	.366	2.60173	1.500	4.662	PER IMP= 86.50
*S TOTAL SURFACE FLOW AT INLET 37										
ADD HYD	49.00	3& 1	3	.01888	48.06	2.070	2.05606	1.500	3.977	
*S DIVIDE FLOW 37 BY RATIO TO MEET STREET CAPACITY										
*S 90% TO STORM SEWER - 10% REMAINS ON SURFACE										
DIVIDE HYD	4.10	3	4	.01699	43.25	1.863	2.05606	1.500	3.977	
	37.10	AND	3	.00189	4.81	.207	2.05605	1.500	3.977	
*S STORM SEWER FLOW FROM INLET 37 TO AP-37										
ROUTE	37.P	4	41	.01699	42.85	1.863	2.05549	1.500	3.940	
*S ADD SURFACE FLOWS FROM BASIN 1 AND 17 AND ROUTE TO AP-37										
COMPUTE NM HYD	1.00	-	1	.00089	2.33	.101	2.11785	1.500	4.084	PER IMP= 62.50
COMPUTE NM HYD	17.00	-	2	.00138	4.09	.189	2.56969	1.500	4.632	PER IMP= 85.00
*S TOTAL SURFACE FLOW FROM BASINS 1 AND 17 AT AP-37										
ADD HYD	17.10	1& 2	2	.00227	6.42	.290	2.39254	1.500	4.417	
*S TOTAL STREET FLOW AP-37										
ADD HYD	37.10	3& 2	3	.00416	11.22	.497	2.23975	1.500	4.217	
*S ADD PIPE FLOW FROM INLET 37 TO MAIN PIPE FLOW										
ADD HYD	37.20	40&41	40	.14213	329.98	14.170	1.86926	1.533	3.628	
*S TOTAL STORM SEWER FLOW ENTERING CITY STORM SEWER AT AP-37										
ADD HYD	40.F	40& 3	40	.14629	340.72	14.666	1.87979	1.533	3.639	
ADD HYD	40.F1	40&90	4	.55480	850.22	67.877	2.29396	1.567	2.394	
*S 108" DIAM; 1100 FT; S=0.007; Q(CAP)=1045 CFS										
ROUTE	40.F12	4	5	.55480	846.97	67.835	2.29256	1.600	2.385	
COMPUTE NM HYD	302.00	-	1	.15398	372.00	15.833	1.92802	1.500	3.775	PER IMP= 50.00
COMPUTE NM HYD	303.00	-	2	.03205	57.93	3.157	1.84707	1.600	2.824	PER IMP= 50.00
ADD HYD	303.10	1& 2	3	.18603	420.49	18.991	1.91408	1.533	3.532	
ADD HYD	P303.2	3& 5	4	.74083	1227.62	86.826	2.19752	1.567	2.589	
*S 108" DIAM; 550 FT; S=0.007; Q(CAP)=1045 CFS										
ROUTE	P303.22	4	5	.74083	1225.88	86.802	2.19690	1.567	2.586	
COMPUTE NM HYD	401.00	-	1	.01945	51.68	2.317	2.23336	1.500	4.152	PER IMP= 70.00
*S 48" DIAM; 1640 FT; S=0.010; Q(CAP)=336 CFS										
ROUTE	P401.02	1	2	.01945	48.91	2.314	2.23050	1.533	3.929	
COMPUTE NM HYD	402.00	-	1	.06011	69.28	4.130	1.28840	1.667	1.801	PER IMP= 21.00
ADD HYD	P402.1	1& 5	3	.80094	1282.72	90.932	2.12871	1.567	2.502	
COMPUTE NM HYD	601.00	-	1	.01639	37.12	1.951	2.23194	1.567	3.538	PER IMP= 70.00
ADD HYD	P601.1	1& 2	4	.03584	85.83	4.265	2.23116	1.533	3.742	
*S TOTAL FLOW TO PROPOSED DETENTION BASIN										
ADD HYD	P601.2	3& 4	5	.83678	1366.89	95.197	2.13310	1.567	2.552	
*S ELEV 5464 IS INVERT OF 48" RCP										
*S ELEV 5473 IS ELEV OF DET POND										
ROUTE RESERVOIR	601.RES	5	2	.83678	223.17	95.248	2.13425	2.433	.417	AC-FT= 47.800

*S

*S	OUTFLOW(CFS)	STORAGE(AC-FT)	ELEV(FT)
*S	0	0.0	5467
*S	10	0.005	5468
*S	24	0.010	5469
*S	48	0.015	5470
*S	72	0.020	5471
*S	94	0.025	5472
*S	112	0.03	5473
*S	130	0.04	5474
*S	143	0.23	5475
*S	155	0.91	5476
*S	165	2.69	5477
*S	175	6.42	5478
*S	185	11.75	5479
*S	194	17.85	5480

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
*S	203	25.04		5481						
*S	211	33.65		5482						
*S	219	42.86		5483						
*S	227	52.33		5484						
*S	235	62.06		5485						
*S										
*S 66" DIAM; 1200 FT; S=0.01; Q(CAP)=336 CFS										
ROUTE	P601.22	2	3	.83678	223.16	95.203	2.13325	2.433	.417	
COMPUTE NM HYD	603.00	-	1	.01761	42.05	2.459	2.61813	1.567	3.731	PER IMP= 90.00
*S 48" DIAM; 950 FT; S=0.01; Q(CAP)=336 CFS										
ROUTE	P603.02	1	2	.01761	41.83	2.457	2.61599	1.600	3.712	
COMPUTE NM HYD	701.00	-	1	.02164	57.50	2.578	2.23336	1.500	4.152	PER IMP= 70.00
ADD HYD	P701.1	1& 2	4	.03925	91.28	5.035	2.40503	1.533	3.634	
ADD HYD	P701.2	4& 3	2	.87603	270.62	100.238	2.14542	1.567	.483	
COMPUTE NM HYD	702.00	-	1	.06609	167.36	9.232	2.61903	1.567	3.957	PER IMP= 90.00
*S TOTAL PIPE FLOW AT EUBANK AND ELIZABETH (APPROX Q AT STA 70+92)										
ADD HYD	P702.1	1& 2	3	.94212	437.98	109.469	2.17865	1.567	.726	
*S 84" DIAM; 1400 FT; S=0.0025; Q(CAP)=320 CFS										
ROUTE	P702.12	3	10	.94212	427.07	109.371	2.17669	1.600	.708	
*S*** BASIN 605-REVISED ****										
COMPUTE NM HYD	605R	-	65	.02906	81.59	3.912	2.52382	1.500	4.387	PER IMP= 85.00
*S** ADD SUB-BASIN 605-R TO P702.12 ***										
*S** STATION 66+23 AT DEVELOPMENT AND EUBANK										
ADD HYD	605.10	65&10	67	.97118	489.88	113.283	2.18708	1.567	.788	
*S ROUTING 606.2 IN PROPOSED 96" THRU SUB-BASIN 607										
*S Q(CAP) = 600 CFS AT SLOPE=0.005										
ROUTE	605.12	67	71	.97118	488.23	113.256	2.18656	1.600	.785	
*S*** BASIN 606A REVISED ****										
COMPUTE NM HYD	606AR	-	66	.02906	68.04	3.430	2.21291	1.533	3.658	PER IMP= 69.00
S ADD PIPE HYD 605.12 TO SUB BASIN 606-R *										
S STATION 61+59 AT RESEARCH AND EUBANK										
ADD HYD	606.10	71&66	73	1.00024	553.85	116.686	2.18733	1.567	.865	
S ROUTE PIPE HYD 606.1 THRU SUB-BASIN 607 REVISED **										
*S ROUTING 606.1 IN PROPOSED 96" THRU SUB-BASIN 607										
*S Q(CAP) = 600 CFS AT SLOPE=0.005										
ROUTE	606.12	73	71	1.00024	548.93	116.620	2.18609	1.600	.858	
COMPUTE NM HYD	607AR	-	65	.03031	78.22	3.610	2.23316	1.500	4.032	PER IMP= 70.00
ROUTE RESERVOIR	607AR.RES	65	2	.03031	27.33	3.591	2.22149	1.800	1.409	AC-FT= 1.367
S ADD PIPE HYD 606.12 TO SUB-BASIN 607AR **										
S PARTIAL FLOW AT STATION 50+68										
ADD HYD	607.10	2&71	72	1.03055	571.74	120.211	2.18713	1.600	.867	
COMPUTE NM HYD	604R	-	65	.06718	124.75	7.299	2.03705	1.600	2.902	PER IMP= 60.00
*S ROUTING 604R THRU BASIN 606R THRU 42" PIPE										
ROUTE	604R.1	65	66	.06718	122.38	7.292	2.03508	1.633	2.846	
*S*** BASIN 606R REVISED ****										
COMPUTE NM HYD	606R	-	67	.05453	88.95	6.423	2.20852	1.700	2.549	PER IMP= 69.00
S ADD 606R TO 604R.1 (STORM DRAIN FLOW) **										
ADD HYD	606R.1	66&67	68	.12171	207.84	13.715	2.11278	1.667	2.668	
ROUTE RESERVOIR	606R.RES	68	2	.12171	101.63	13.653	2.10338	2.000	1.305	AC-FT= 4.446
*S ROUTING 604R.1 THRU BASIN 607R THRU 42" PIPE "A"										
ROUTE	606R.12	2	69	.12171	101.55	13.646	2.10227	2.000	1.304	
COMPUTE NM HYD	607R	-	70	.06359	118.52	7.562	2.22960	1.633	2.912	PER IMP= 70.00
ROUTE RESERVOIR	607R.RES	70	2	.06359	52.34	7.522	2.21785	2.000	1.286	AC-FT= 2.617
S ADD 607R TO 606R.12 (STORM DRAIN FLOW) **										
ADD HYD	607R.1	69& 2	50	.18530	153.89	21.168	2.14194	2.000	1.298	
*S ROUTING 607R.1 THRU BASIN 608R THRU 48" PIPE "B"										
ROUTE	607R.12	50	51	.18530	153.58	21.149	2.14003	2.033	1.295	
S TOTAL FLOW AT STATION 50+68										
ADD HYD	607AR.1	72&51	71	1.21585	626.70	141.360	2.17995	1.633	.805	
S ROUTE PIPE HYD 607.1 THRU SUB-BASIN 608 REVISED **										
*S ROUTING 607.1 IN PROPOSED 96" ; S=0.0082 FT/FT										
ROUTE	607.12	71	72	1.21585	626.04	141.310	2.17919	1.633	.805	
*S*** BASIN 608 REVISED****										
COMPUTE NM HYD	608R	-	65	.04000	36.91	1.632	.76501	1.633	1.442	PER IMP= .00
S ADD 608R TO 607R.12 (STORM DRAIN FLOW) **										
S STATION 42+23										
ADD HYD	608R.1	72&65	52	1.25585	662.95	142.942	2.13415	1.633	.825	
S ROUTE PIPE HYD 608R.1 THRU SUB-BASIN 609 REVISED **										
*S ROUTING IN PROPOSED 96" ; S=0.0082 FT/FT										
ROUTE	609.13	52	74	1.25585	660.65	142.887	2.13332	1.633	.822	
*S*** BASIN 609 REVISED****										

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
COMPUTE NM HYD	609R	-	69	.05375	57.31	5.270	1.83839	1.866	1.666	PER IMP= 50.00
ROUTE RESERVOIR	609R.RES	69	2	.05375	42.99	5.258	1.83418	2.133	1.250	AC-FT= 1.075

*S*** ADD HYD 609 TO PIPE FLOW 609AR.13 **
S STATION 33+00
*S*** THIS WILL BE TOTAL FLOW TO PROPOSED OUTFALL CHANNEL

ADD HYD	609.10	2&74	80	1.30960	673.25	148.145	2.12104	1.667	.803

FINISH

APPENDIX "B"

Pipe	Upstream Node	Downstream Node	Inlet C Coefficient	Inlet CA (acres)	stream GL (ft)	Average Velocity (ft/s)
79+86 TO 79+94	79+94	79+86	0.00	0.00	162.64	11.24
76+34 TO 79+86	79+86	76+34	N/A	N/A	161.78	10.14
73+46 TO 76+34	76+34	73+46	N/A	N/A	161.65	8.41
73+08 TO 73+46	73+46	73+08	N/A	N/A	161.55	7.57
72+84 TO 73+08	73+08	72+84	0.00	0.00	160.98	12.29
71+00 TO 72+84	72+84	71+00	N/A	N/A	160.39	12.03
70+92 TO 71+00	71+00	70+92	0.00	0.00	160.18	12.03
70+84 TO 70+92	70+92	70+84	0.00	0.00	159.84	13.32
66+51 TO 70+84	70+84	66+51	N/A	N/A	157.80	13.21
66+23 TO 66+51	66+51	66+23	N/A	N/A	157.76	12.97
62+20 TO 66+23	66+23	62+20	0.00	0.00	155.10	13.81
61+59 TO 62+20	62+20	61+59	N/A	N/A	154.85	12.26
61+35 TO 61+59	61+59	61+35	0.00	0.00	154.08	14.63
57+19 TO 61+35	61+35	57+19	N/A	N/A	151.38	14.88
53+76 TO 57+19	57+19	53+76	N/A	N/A	149.34	14.86
50+80 to 53+76	53+76	50+80	N/A	N/A	148.26	14.09
50+68 to 50+80	50+80	50+68	N/A	N/A	148.28	12.69
50+36 TO 50+68	50+68	50+36	0.00	0.00	147.68	14.91
46+40 TO 50+36	50+36	46+40	N/A	N/A	145.05	15.25
42+23 TO 46+40	46+40	42+23	N/A	N/A	143.39	14.28
42+15 TO 42+23	42+23	42+15	0.00	0.00	143.02	15.24
37+25 TO 42+15	42+15	37+25	N/A	N/A	139.84	15.54
33+00 TO 37+25	37+25	33+00	N/A	N/A	137.82	14.87
32+40 TO 33+00	33+00	32+40	0.00	0.00	137.05	15.49
30+30 TO 32+40	32+40	30+30	N/A	N/A	135.38	15.86
26+82 TO 30+30	30+30	26+82	N/A	N/A	134.59	14.45
26+58 TO 26+82	26+82	26+58	N/A	N/A	134.51	13.64
23+02 TO 26+58	26+58	23+02	0.00	0.00	131.43	16.69
18+52 TO 23+02	23+02	18+52	N/A	N/A	128.46	16.69
17+70 TO 18+52	18+52	17+70	N/A	N/A	127.93	16.68
17+22 TO 17+70	17+70	17+22	N/A	N/A	120.90	21.46
17+02 TO 17+22	17+22	17+02	N/A	N/A	119.75	22.56
16+82 TO 17+02	17+02	16+82	N/A	N/A	118.11	28.34
14+80 TO 16+82	16+82	14+80	N/A	N/A	80.35	27.36
13+77 TO 14+80	14+80	13+77	N/A	N/A	76.39	17.85
12+60 TO 13+77	13+77	12+60	N/A	N/A	75.20	17.50
5+60 TO 12+60	12+60	5+60	N/A	N/A	68.11	17.50
3+70 TO 5+60	5+60	3+70	N/A	N/A	66.18	17.50

NOTES

PROPOSED SOUTH EUBANK STORM DRAIN (COA PROJECT)

DIRECTION OF FLOW

UNDEVELOPED PROPERTIES (& EXISTING M.H. PARK) W/100-YR PEAK FLOW RESTRICTED TO 1.3 CFS/AC

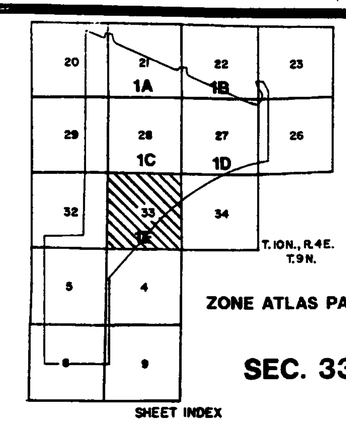
FUTURE STORM DRAIN NEED FOR FUT. DEVELOP. (NOT COA PROJECT)

SUB-BASIN NUMBER
AREA (ACRES)
% IMPERVIOUS
100-YEAR RUNOFF VOLUME (AC-FT)
100-YEAR PEAK RUNOFF FLOW (CFS)

SUB-BASIN DATA

* = 100 YR PEAK FLOW AFTER DETENTION (CFS)

BASIN BOUNDARY



N

1"=500'

2' CONTOUR INTERVAL

ZONE ATLAS PAGE M-21

SEC. 33

THE AERIAL PHOTOGRAPH ON THIS SHEET HAS BEEN RECTIFIED TO SELECTED PLANIMETRIC DETAIL OBTAINED DURING MAP COMPILATION. CONSEQUENTLY, THE PHOTO IMAGES FALLING OUTSIDE OF THE MAPPING AREA ARE UNCONTROLLED.

500 250 0 500 1000

DATE OF PHOTOGRAPHY: MARCH 2, 1994

ADDENDUM #2 TO SOUTH EUBANK AREA

STORM DRAINAGE ANALYSIS (NOV 1995)

SMITH ENGINEERING COMPANY

MAP M-21 PLATE 1E

City of Albuquerque - Public Works Department/Hydrology Division

SO. EUBANK STORM DRAINAGE ANALYSIS

AMMENDMENT #2

NOVEMBER 2, 1998

SMITH ENGINEERING, INC.

City of Albuquerque - Public Works Department/Hydrology Division

SO. EUBANK STORM DRAINAGE ANALYSIS

AMMENDMENT #1

FEBRUARY 2, 1998
SMITH ENGINEERING, INC.

Smith Engineering Company
A Full Service Engineering Company

February 2, 1998

Loren D. Meinz
Civil Engineer
City of Albuquerque Public Works
Hydrology Division
P.O. Box 1293
Albuquerque, NM 87103

Re: Amendment #1 to the South Eubank Area Storm Drainage Analysis (November 1995)

Dear Mr. Meinz:

Smith Engineering Company is pleased to submit to your office an amendment to the hydrology for the South Eubank Area Storm Drainage Analysis (November 1995). This Amendment #1 updates the hydrology that contributes to the proposed Eubank Storm Drain between Cochiti Road and Bell Avenue. A total drainage area of about 86 acres has been reanalyzed based on field inspections and hydraulic analysis of the existing Eubank Storm Drain System. The following paragraphs describe this analysis.

Hydrologic Assumptions

Five sub-basins have been delineated that contribute runoff to Eubank Boulevard between Cochiti Road and Bell Avenue. These sub-basins are 506, 404, 405B, 405AA, and 405AB. These sub-basins are shown on Plate 1C. Sub-basin 506 represents Southern Avenue from Elizabeth Street to Eubank Boulevard. This flow is assumed to flow north on Eubank. Sub-basin 405B flows west and south into sub-basin 404. The combined flow from 404 collects at the northwest corner of this sub-basin where it presently flows north in Eubank and then west down Acoma. Sub-basins 405AA and 405AB represent the drainage at the Home Depot as described in the "Drainage Report for Albuquerque II Home Depot" (April 28, 1994). The modeling of the Home Depot limits the outflow from this site to a 100-year peak flow of 3 cfs as described in their drainage report. The hydrologic analysis for this amendment analyzed the 100-year 6-hour event (also modeled in the original "South Eubank Area Storm Drainage Analysis"). All sub-basins south of Southern Avenue will not enter our project area. They will either flow west in a future Southern Storm Drain System or south in a future South Eubank Storm Drain System (as described in the "Manzano Mesa Master Drainage Study", August 1996). All hydrologic analysis was performed for future developed conditions with Option B in place. Option B assumes that the future South Eubank Storm Drain System that will extend down Elizabeth Street will intercept flow from the east of the study area.

Changes to Option B

The following changes were made to the AHYMO Option B model (Optionb.dat) used in the "South Eubank Area Storm Drainage Analysis" report.

- Based on the "Drainage Report for Albuquerque II Home Depot" a portion of sub-basin 406 was added to sub-basin 405AB.
- Sub-basin 405 was split into 405AA, 405AB, and 405B based on information obtained from field visits and the "Drainage Report for Albuquerque II Home Depot".
- Sub-basin 405B was modeled with a land treatment of 90% "D".

6400 Uptown Boulevard, N.E. Suite 500E Albuquerque, N.M. 87110 Telephone 505/884-0700 Fax 505/884-2376

- A future detention basin has been modeled at the northwest corner of sub-basin 404 in order to restrict the 100-year peak flows entering the existing and proposed Eubank Storm Drain, and in order to eliminate runoff flowing west down Acoma.

Analysis of Storm Drain Capacity

An analysis of the existing Eubank Storm Drain System (from Chico to Cochiti) and the proposed Eubank Storm Drain System (Cochiti to Bell) was performed using the program, StormCAD v 1.5. This program evaluates the capacity of these storm drains based on the hydraulic grade line. A schematic of this storm drain system is shown in Figure 1. The output from StormCAD is shown in Appendix A.

The results from StormCAD show that the existing Eubank Storm Drain (from Chico to Cochiti) has a capacity of about 160 cfs. Option B shows that about 50 cfs will be flowing west down the Central Avenue Storm Drain and joining the Eubank Storm Drain. This results in a capacity of only about 110 cfs for the Eubank Storm Drain from Central to Bell. This StormCAD model was started with an HGL at street level at Chico due to the fact that Option B predicts flow out of the manhole at Chico/Eubank during the 100-year event.

The "South Eubank Area Storm Drainage Analysis" estimated the capacity of the Eubank Storm Drain from Central to Bell at 140 cfs. In fact, our updated analysis shows that the capacity of this system is only 110 cfs.

Hydrologic Analysis

The AHYMO model, Optionb.dat, was modified as described in the previous paragraphs. A drainage map showing the updated sub-basins is shown on Plate 1C. The updated 100-year 6-hour AHYMO model, Eubsdb.dat, is shown as a schematic in Plate 5B. A 10-year 6-hour analysis was also performed using the AHYMO model, Eubsdb10.dat. The summary output for the 100-year event is included in Appendix B.

The results of this updated analysis show that about 32 cfs is generated by sub-basin 506 for the 100-year event. This flow turns north onto Eubank Boulevard. Sub-basin 405B produces a 100-year peak flow of 75 cfs that has historically flowed south into sub-basin 404. The combined peak flow at the northwest corner of 404 is 153 cfs. A proposed detention basin was modeled at this location to reduce this peak flow so as not to exceed the capacity of the proposed Eubank Storm Drain (Cochiti to Bell) which is 110 cfs. A summary of the hydrologic parameters used in this amendment is shown in Table 1. A summary of the peak flows at the analysis points is shown in Table 2.

TABLE 1. SUMMARY OF BASIN PARAMETERS

Basin	Area (ac)	Tp (hrs)	Land Treatment (%) A	B	C	D	100-Yr 6-Hr Peak Q (cfs)	100-Yr 6-Hr Vol (ac-ft)
404	31	0.26	10	20	0	70	90	4.8
405B	17	0.15	0	10	0	90	75	3.1
405AA	5	0.13	0	10	0	90	24	0.9
405AB	11	0.13	0	10	0	90	52	2.0
506	10	0.21	10	15	0	75	32	1.5
406*	12	0.20	1	0	4	95	45	2.2

* Sub-basin 406 does not contribute to the study area, but a portion of its original area was transferred to sub-basin 405AB.

TABLE 2. SUMMARY OF PEAK FLOWS AND RUNOFF VOLUMES AT ANALYSIS POINTS

Analysis Point	Description	100-yr 6-hr Peak Q Overland (cfs)	100-yr 6-hr Peak Q in Storm Drain (cfs)
29	Flow west on Acoma	0	N/A
29A	Flow just south of Eubank/Bell	30	N/A
29B	Flow at NW corner of 404 upstream of proposed detention basin	153	N/A
29C	Flow just north of Eubank/Bell	0	104
29D	Flow from Home Depot SD	N/A	3
29E	Flow just south of Eubank/Central (Capacity is 110 cfs +/-)	0	106*

* Total flow in Eubank Storm Drain is sum of routed flow from sub-basin 506 (30 cfs), restricted flow from NW corner of sub-basin 404 (87 cfs), and restricted flow from Home Depot (3 cfs). Due to differences in timing and effects of routing this total flow is 106 cfs.

Recommendations

It is important to insure that as development proceeds in the study area that the capacity of the existing and proposed storm drains will not be exceeded. The following recommendations are made based on the updated hydrology summarized in the previous paragraphs:

- Collect the 100-year peak flow of 32 cfs from sub-basin 506 (Southern Blvd. Flow) near the northwest corner of sub-basin 404.
- Reduce the 100-year peak flow generated by sub-basin 404 (including the historic flow contributed by sub-basin 405B) from 153 cfs to 87 cfs. This would require a detention basin with an approximate volume of 2.6 acre-feet.
- Restrict the 100-year peak flow from Home Depot (sub-basins 405AA and 405AB) to 3 cfs as presented in the "Drainage Report for Albuquerque II Home Depot".
- Reduce the 100-year discharge that flows west on Acoma to 0 cfs.

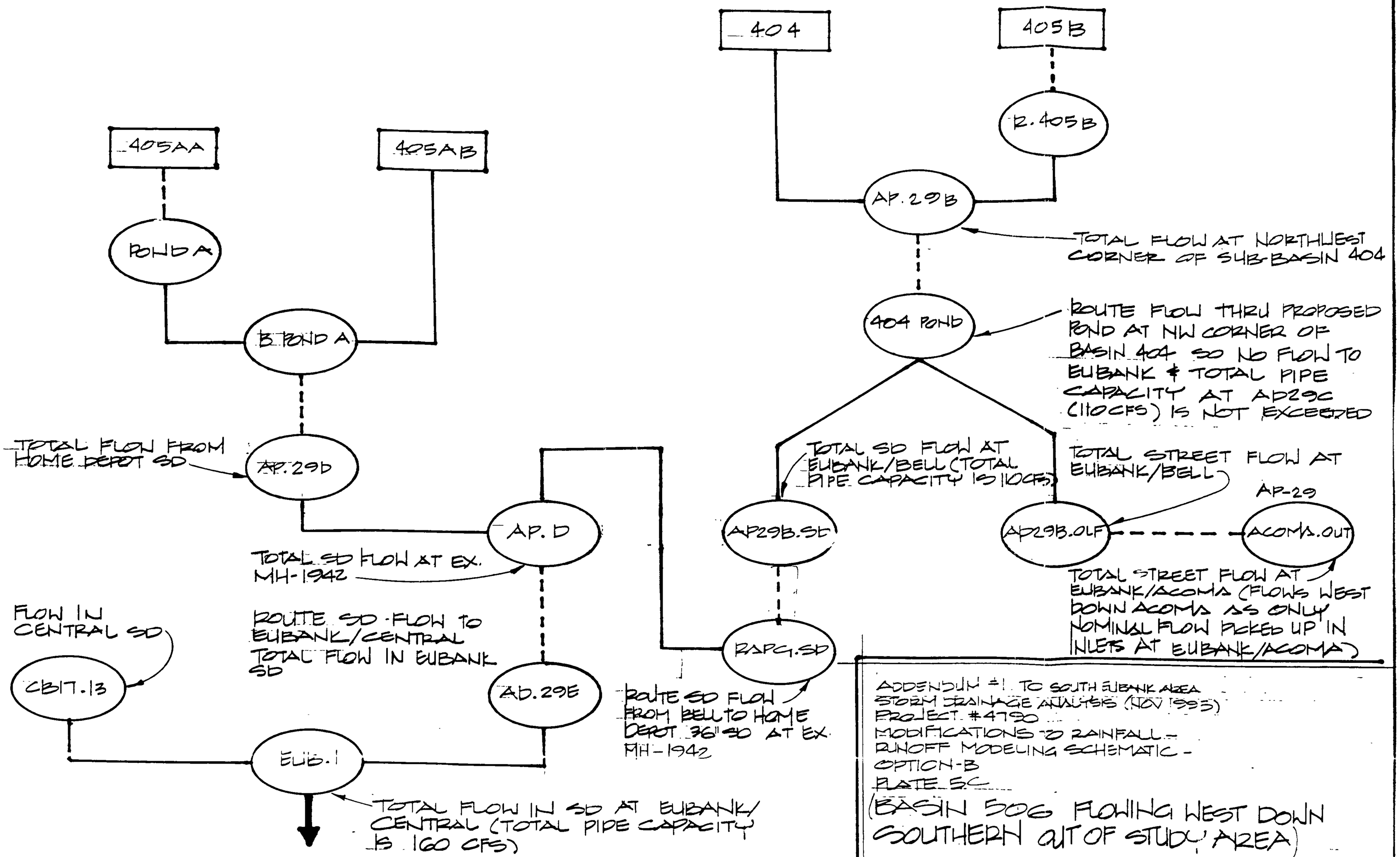
Should you have any questions or comments about this analysis please call Steve Kemna at (505) 884-0700.

Sincerely,

Smith Engineering Company

Stephen Paul Kemna

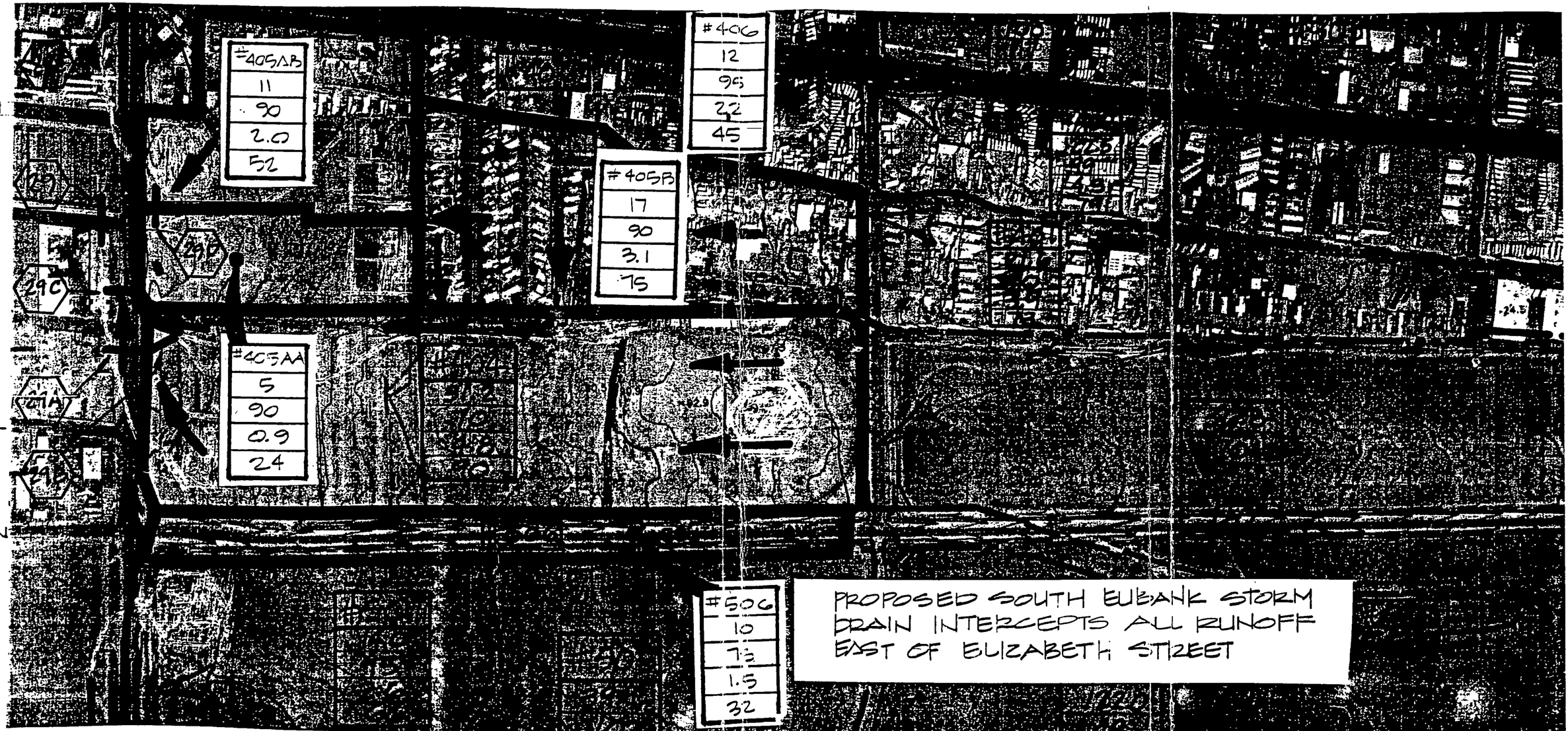
Stephen P. Kemna, P.E.



ADDENDUM #1 TO SOUTH EUBANK AREA STORM DRAINAGE ANALYSIS (NOV 1995)
PROJECT #4790
MODIFICATIONS TO RAINFALL-RUNOFF MODELING SCHEMATIC - OPTION-B
PLATE 5C

(BASIN 506 FLOWING WEST DOWN SOUTHERN OUT OF STUDY AREA)

SMITH ENGINEERING COMPANY



LEGEND

DIRECTION OF OVERLAND FLOW

34 — ANALYSIS POINT NUMBER WITH LOCATION BAR

BASIN BOUNDARY

SUB-BASIN NUMBER
AREA (ACRES)
% IMPERVIOUS
100-YEAR RUNOFF VOLUME (AC-FT)
100-YEAR PEAK RUNOFF FLOW (CFS)

SUB-BASIN DATA

300 150 0 300

1" = 300'

ADDENDUM # TO SOUTH EUBANK AREA-STORM DRAINAGE ANALYSIS (NOV. 1995) PROJ. # 4190

PLATE 1C

MAP 1-21

SMITH ENGINEERING COMPANY

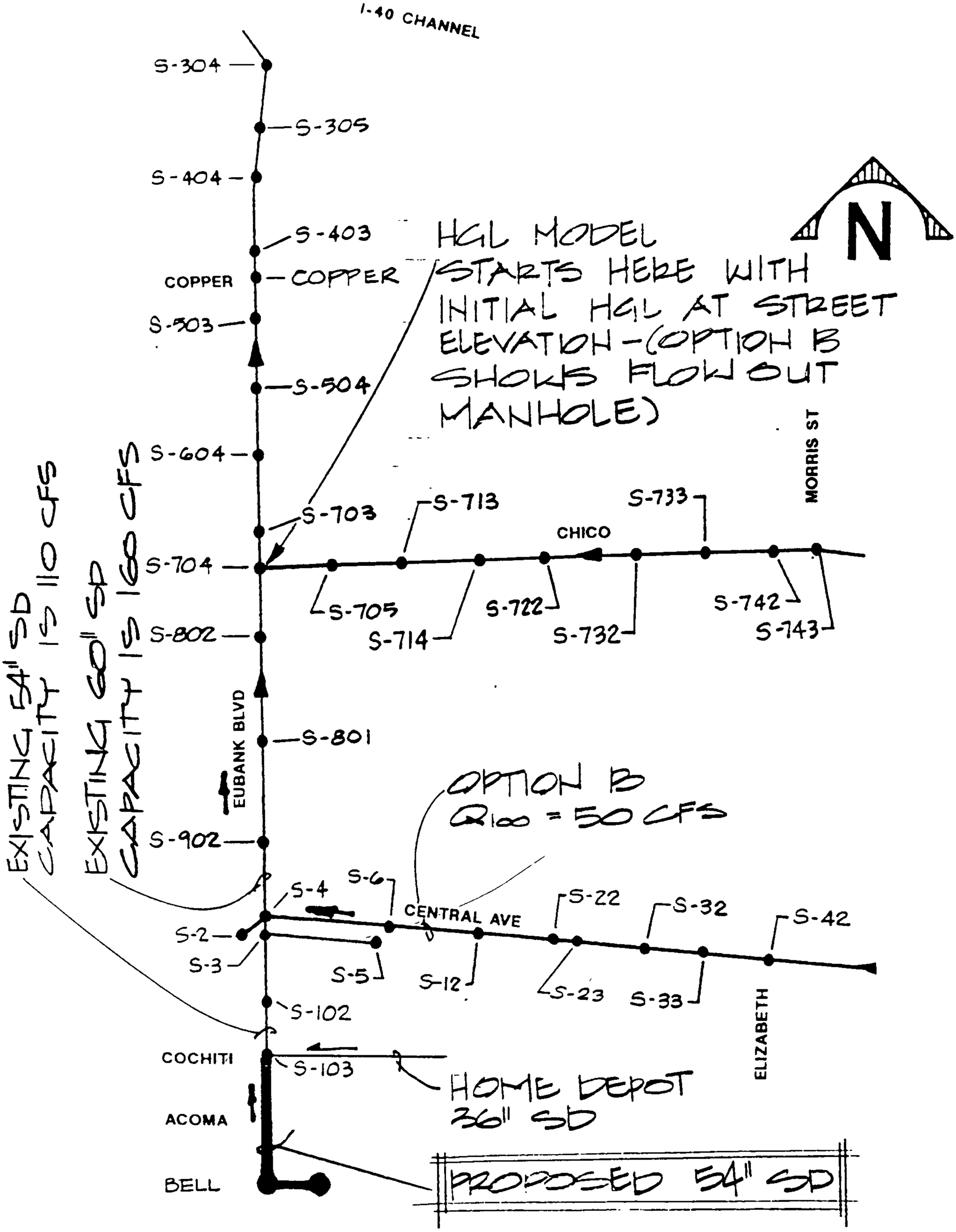
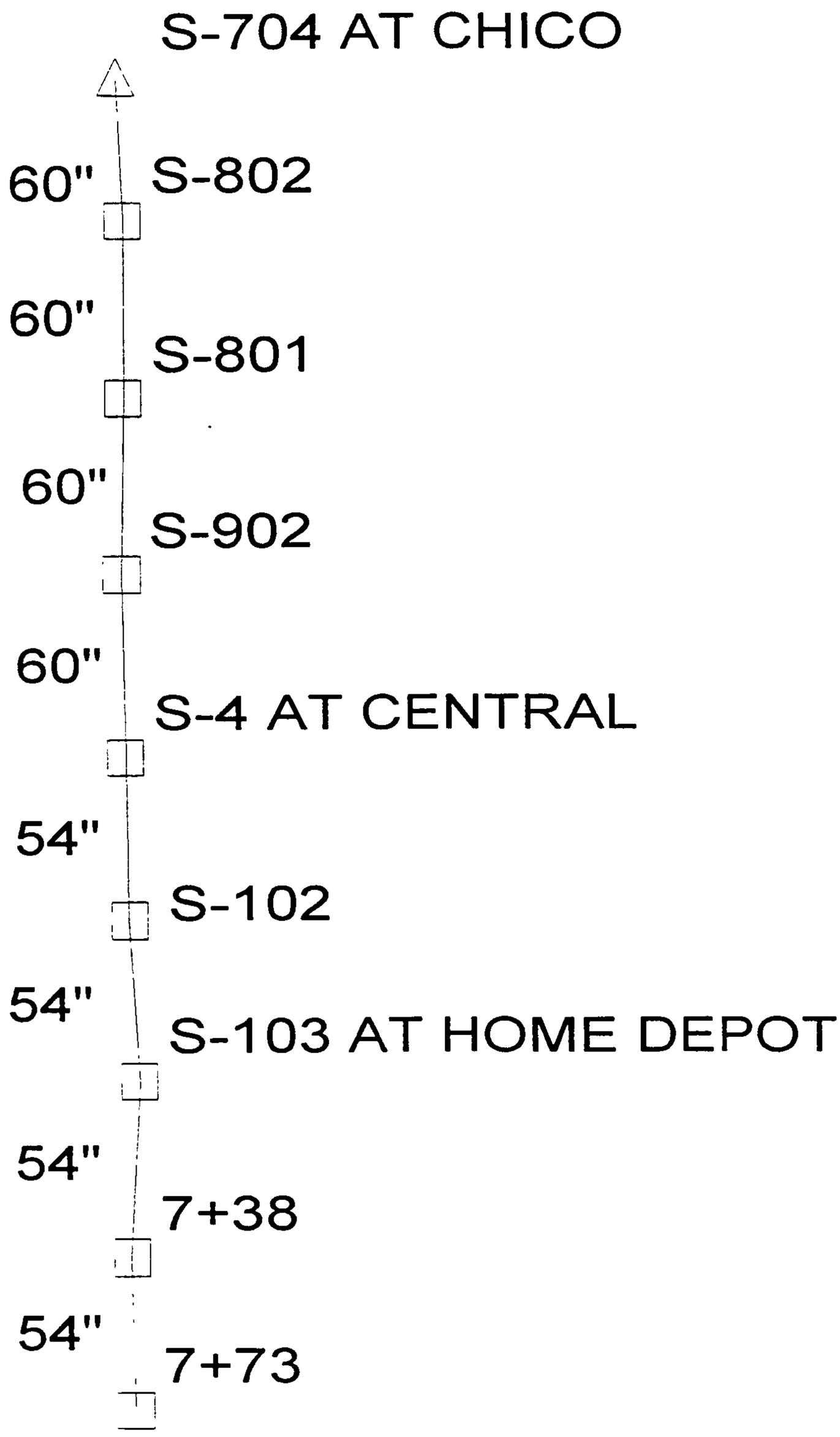


FIGURE 1
SCHEMATIC OF COA SD SYSTEM

SMITH ENGINEERING COMPANY

APPENDIX A


S-704 AT CHICO
60"
S-802
60"
S-801
60"
S-902
60"
S-4 AT CENTRAL
54"
S-102
54"
S-103 AT HOME DEPOT
54"
7+38
54"
7+73

Pipe Report

Pipe	Upstream Node	Downstream Node	Discharge (cfs)	Length (ft)	Constructed Slope (ft/ft)	Section Size	Roughness	Capacity (cfs)	Upstream Invert Elevation (ft)	Downstream Invert Elevation (ft)	Upstream Ground Elevation (ft)	Downstream Ground Elevation (ft)	Upstream Cover (ft)	Downstream Cover (ft)	Upstream HGL (ft)	Downstream HGL (ft)	Upstream Depth (ft)	Downstream Depth (ft)	Average Velocity (ft/s)
54"	7+73	7+38	110 00	74 00	0 005000	54 inch	0 013	139 04	5,456 07	5,455 70	5,464 70	5,464 20	4.13	4 00	5,464 30	5,464 07	8 23	8 37	6 92
54"	7+38	S-103 AT HOME DEPO	110 00	445 00	0 004989	54 inch	0 013	138 89	5,455 60	5,453 38	5,464 20	5,463 55	4.10	5 67	5,464 03	5,462 64	8 43	9 26	6 92
54"	S-103 AT HOME DEPOT	S-102	110 00	238 00	0 005084	54 inch	0 013	140 21	5,453 38	5,452 17	5,463 55	5,468 00	5 67	11 33	5,462 53	5,461 78	9 15	9 61	6 92
54"	S-102	S-4 AT CENTRAL	110 00	290 00	0 006483	54 inch	0 013	158 32	5,452 07	5,450 19	5,468 00	5,465 00	11 43	10 31	5,461 75	5,460 84	9 68	10 65	6 92
60"	S-4 AT CENTRAL	S-902	160 00	470 00	0 005000	60 inch	0 013	184 15	5,449 72	5,447 37	5,465 00	5,463 70	10 28	11 33	5,460 68	5,458 91	10 96	11 54	8 15
60"	S-902	S-801	160 00	490 00	0 005306	60 inch	0 013	189 70	5,447 22	5,444 62	5,463 70	5,459 50	11 48	9 88	5,458 86	5,457 01	11 64	12 39	8 15
60"	S-801	S-802	160 00	372 00	0 005376	60 inch	0 013	190 96	5,444 62	5,442 62	5,459 50	5,457 00	9 88	9 38	5,456 96	5,455 55	12 34	12 93	8 15
60"	S-802	S-704 AT CHICO	160 00	265 00	0 004943	60 inch	0 013	183 11	5,442 47	5,441.16	5,457.00	5,454 50	9 53	8 34	5,455 50	5,454 50	13 03	13 34	8 15

APPENDIX B


```
AHYMO SUMMARY TABLE (AHYMO194) - AMAFCA Hydrologic Model - January, 1994          RUN DATE (MON/DAY/YR) =01/28/1998
INPUT FILE = EUBSDB.DAT                                                                   USER NO.= T_DALTON.S94

                                FROM  TO                    PEAK        RUNOFF                TIME TO   CFS      PAGE =   1
                   HYDROGRAPH   ID    ID        AREA     DISCHARGE     VOLUME      RUNOFF      PEAK    PER
COMMAND          IDENTIFICATION NO.   NO.     (SQ MI)     (CFS)       (AC-FT)    (INCHES)   (HOURS)   ACRE      NOTATION

*S       INCLUDES PROPOSED 96" PIPE AND DETENTION BASIN WITH 66" OUTLET PIPE
*S       THAT PICK UP FLOWS FROM BASIN 501, 502, 503, 504, 401, 403
*S       DOES PICK UP PIPE FLOW FROM CENTRAL AND ELIZABETH
*S       INCREASE PIPE SIZE IN JANE STREET AND DIVERTS THIS FLOW NORTH WARD
*S       TO I-40 DIVERSION CHANNEL
*S       TAKE FLOW FROM 54" STUB-OUT AT CENTRAL AND EUBANK TO EUBANK LINE
*S
START                                                                                                  TIME=       .0
RAINFALL  TYPE= 1                                                                                      RAIN6=    2.56
*S  100-YEAR STORM
*S  ONE-HOUR=2.07
*S  SIX-HOUR=2.56
*S HYD 101 IS OLF THAT IS AT THE BEGINNING OF THE CENTRAL SD SYSTEM
COMPUTE NM HYD        101.00  -    11        .03319      103.12       4.092     2.31187    1.500    4.855 PER IMP= 100.0
*S ROUTING 101 THRU 102 ON CENTRAL AVENUE'S NORTH SIDE
ROUTE                 101.02  11   12        .03319       91.32       4.092     2.31188    1.533    4.299
COMPUTE NM HYD        102.00  -    11        .05253      149.46       5.978     2.13387    1.500    4.446 PER IMP=  88.0
ADD HYD               102.10 11&12 13        .08572      237.16      10.071     2.20279    1.533    4.323
*S DIVIDE BETWEEN MH# S-132 TO MH# S-262
*S****     PIPE HYD CB1 AND OLF HYD 102.13 IS AP #9     *****
DIVIDE HYD               CB1  13   11        .05140       61.00       6.039     2.20279    1.367    1.854
                      102.13  AND  12        .03432      176.16       4.032     2.20279    1.533    8.020
*S OLF HYD 102.13 THRU 103 ON CENTRAL STREET'S NORTH SIDE
ROUTE                 102.12  12   14        .03432      137.98       4.032     2.20293    1.600    6.282
COMPUTE NM HYD        103.00  -    13        .02347       49.09       2.453     1.95937    1.600    3.268 PER IMP=  77.0
ADD HYD               103.13 13&14 12        .05779      187.07       6.485     2.10399    1.600    5.058
*S DIVIDE CATCH BASINS AND CATTLE GUARD BETWEEN MH# S-102  TO  MH# S-132
*S OLF HYD 103.03 IS AT JUAN TABO AND CENTRAL ON THE NORTH SIDE
*S****     OLF HYD 103.03 IS AP #13     *****
DIVIDE HYD               CB2  12   14        .04575       99.00       5.133     2.10399    1.500    3.381
                      103.03  AND  15        .01204       88.07       1.351     2.10399    1.600   11.427
*S INCLUDES SUB-BASIN 104 AND 105
COMPUTE NM HYD        105.00  -    16        .06765      170.33       7.502     2.07926    1.533    3.934 PER IMP=  84.0
COMPUTE NM HYD        106.00  -    17        .03425       68.27       3.016     1.65135    1.567    3.115 PER IMP=  48.0
*S ROUTING 106 THRU 105/108 ON SKYLINE ROAD
ROUTE                 106.02  17   18        .03425       61.47       3.016     1.65136    1.633    2.804
COMPUTE NM HYD        108.00  -    13        .04283      111.62       4.599     2.01333    1.533    4.072 PER IMP=  80.0
ADD HYD               106.10 16&18 17        .10190      222.24      10.518     1.93543    1.567    3.408
*S TOTAL FLOW GOING TO JUAN TABO'S CATTLE GUARD
ADD HYD               108.10 13&17 16        .14473      327.77      15.117     1.95848    1.533    3.539
*S CATTLE GUARD AND CATCH BASINS BETWEEN MH# S-993  TO  MH# S-903
*S****     OLF HYD 108.13 IS AP #12     *****
DIVIDE HYD               CB3  16   17        .14318      300.00      14.955     1.95848    1.500    3.274
                      108.13  AND  18        .00155       27.77        .162     1.95847    1.533   27.915
COMPUTE NM HYD        201.00  -    21        .02777       84.21       3.318     2.24012    1.500    4.738 PER IMP=  95.0
*S EXISTING DETENTION BASIN IN BASIN 201
*S ROUTING RESERVOIR WITH SMALL ORIFICE PIPES AND 20' WEIR
ROUTE RESERVOIR      201.RES  21   22        .02777       23.79       3.316     2.23882    1.933    1.339 AC-FT=    2.05
*S
*S       OUTFLOW(CFS)    STORAGE(AC-FT)    ELEV(FT)
*S           0                0              5620
*S           2.5              0.468          5621
*S           3.8              0.937          5622
*S           4.8              1.405          5623
*S           5.6              1.873          5624
*S          29.0              2.107          5624.5
*S          70.0              2.342          5625
*S
*S ROUTING 201.RES THRU 202
ROUTE                 201.02  22   21        .02777       20.59       3.315     2.23821    2.066    1.159
COMPUTE NM HYD        202.00  -    22        .04041      103.90       4.816     2.23453    1.567    4.017 PER IMP=  95.0
COMPUTE NM HYD        209.00  -    25        .05761      119.24       5.400     1.75764    1.567    3.234 PER IMP=  62.0
ADD HYD               202.10 21&22 24        .06818      105.99       8.131     2.23602    1.567    2.429
ADD HYD               209.10 24&25 23        .12579      225.22      13.531     2.01693    1.567    2.798
*S DIVIDE OLF HYD 209.1 BY PROPOSED TOMLINSON PIPE NETWORK
*S****     OLF HYD 209.13 IS AP #1     *****
```

```
DIVIDE HYD          DIVTOM.1   23   71        .11138       125.00      11.981      2.01693    1.467    1.754
                      209.13  AND   24        .01441       100.22       1.550      2.01693    1.567   10.869
*S ROUTE DIVTOM.1 THRU 210 IN 48" AND 54" PIPE
*S Q(CAP) = 208 CFS, AT SLOPE=0.013
```

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	NOTATION
ROUTE	DIVTOM.12	71	72	.11138	125.00	11.981	2.01689	1.766	1.754	
*S ROUTING 209.13 THRU 210 ON TOMLINSON DRIVE TO OUTFALL										
ROUTE	207C.12	24	22	.01441	65.76	1.550	2.01728	1.633	7.131	
COMPUTE NM HYD	203.00	-	21	.08386	172.02	8.709	1.94717	1.600	3.205	PER IMP= 75.0
*S DIVIDE SOUTH SIDE CENTRAL AVENUE BETWEEN MH# S-132 AND MH# S-262(CHECK TO S										
*S**** OLF HYD 203.03 IS AP #10 *****										
DIVIDE HYD	CB5	21	24	.00444	1.00	.461	1.94716	.967	.352	
	203.03	AND	25	.07942	171.02	8.247	1.94716	1.600	3.365	
*S**** PIPE HYD CB5.1 IS AP #10 *****										
ADD HYD	CB5.1	24&11	21	.05584	62.00	6.500	2.18245	1.367	1.735	
*S FROM MH# S-102 TO MH# S-132										
*S Q(CAP) = 73 CFS										
ROUTE	CB5.12	21	27	.05584	62.00	6.500	2.18245	1.966	1.735	
*S ROUTING 203.03 THRU 204 ON CENTRAL AVENUE'S SOUTH SIDE										
ROUTE	203.02	25	23	.07942	152.20	8.248	1.94723	1.667	2.995	
COMPUTE NM HYD	204.00	-	21	.06581	141.97	7.399	2.10798	1.600	3.371	PER IMP= 86.0
ADD HYD	204.10	23&21	24	.14523	287.70	15.646	2.02007	1.633	3.095	
*S DIVIDE SOUTH SIDE CENTRAL AVENUE BETWEEN MH# S-102 AND MH# S-123										
*S OLF HYD 204.13 IS AT JUAN TABO AND CENTRAL ON THE SOUTH SIDE										
*S**** OLF HYD 204.13 IS AP #14 *****										
DIVIDE HYD	CB6	24	25	.10358	117.00	11.160	2.02007	1.467	1.765	
	204.13	AND	86	.04164	170.70	4.486	2.02007	1.633	6.405	
ADD HYD	CB6.1	27&25	21	.15943	179.00	17.660	2.07694	1.966	1.754	
*S TOTAL PIPE FLOW AT JUAN TABO AND CENTRAL										
*S Q(CAP) AT MH# S-102 IS 260 CFS										
*S**** PIPE HYD CB6.2 IS AP #13 AND AP #14 *****										
ADD HYD	CB6.2	14&21	99	.20518	278.00	22.793	2.08297	1.766	2.117	
COMPUTE NM HYD	205.00	-	21	.01633	43.64	1.651	1.89569	1.500	4.175	PER IMP= 71.0
*S ROUTING 205 THRU 206 ON JUAN TABO STREET										
ROUTE	205.02	21	23	.01633	36.77	1.651	1.89570	1.567	3.519	
COMPUTE NM HYD	206.00	-	24	.01425	38.20	1.550	2.03920	1.500	4.189	PER IMP= 81.0
ADD HYD	206.10	23&24	21	.03058	73.94	3.201	1.96254	1.533	3.778	
*S DIVIDE OLF HYD 206.1 AT MH# S-592										
*S PIPE HYD 206.13 GOES TO PROPOSED SURGE POND										
DIVIDE HYD	CB18	21	27	.02644	42.00	2.768	1.96254	1.433	2.482	
	206.13	AND	28	.00414	31.94	.433	1.96254	1.533	12.068	
*S FROM MH# S-791 TO MH# S-592										
*S Q(CAP) = 48 CFS										
ROUTE	CB18.02	27	29	.02644	41.97	2.768	1.96256	1.700	2.480	
*S**** SUB-BASIN 208 IS AP #5 *****										
COMPUTE NM HYD	208.00	-	24	.09685	190.67	9.850	1.90704	1.600	3.076	PER IMP= 72.0
*S DIVIDE SUB-BASIN 208 AT MH# S-692										
DIVIDE HYD	CB19	24	27	.09400	161.00	9.561	1.90703	1.533	2.676	
	208.03	AND	21	.00285	29.67	.290	1.90703	1.600	16.269	
*S ROUTING 208.03 THRU 208 ON JUAN TABO STREET										
ROUTE	208.02	21	24	.00285	22.63	.290	1.90884	1.667	12.411	
*S TOTAL PIPE FLOW AT MH# S-691										
ADD HYD	CB19.1	27&29	23	.12045	202.97	12.329	1.91922	1.700	2.633	
*S DIVIDE PIPE HYD CB19.1 BY 36" INLET PIPE AT MH# S-691										
DIVIDE HYD	CB19.13	23	91	.06513	55.00	6.666	1.91922	1.400	1.320	
	DET.5	AND	92	.05532	147.97	5.662	1.91922	1.700	4.179	
*S TOTAL INFLOW TO DETENTION POND IN SUB-BASIN 502										
ADD HYD	DET.502	28&92	21	.05945	178.96	6.095	1.92223	1.533	4.703	
*S FROM MH# S-691 TO MH# S-792										
*S Q(CAP) = 47 CFS										
ROUTE	CB19.12	91	23	.06513	55.00	6.666	1.91922	1.467	1.320	
*S TOTAL PIPE FLOW AT MH# S-792										
ADD HYD	CB19.2	23&24	28	.06798	77.63	6.956	1.91878	1.667	1.784	
*S FROM MH# S-94(OUTFALL) TO MH# S-792										
*S Q(CAP) = 94 CFS										
ROUTE	CB19.12	28	29	.06798	71.89	6.956	1.91878	1.700	1.652	
COMPUTE NM HYD	207.00	-	23	.03038	70.01	2.857	1.76351	1.533	3.601	PER IMP= 63.0
*S ROUTE 207 THRU 210 IN PROPOSED 48" PIPE										
*S Q(CAP) = 76 CFS, AT SLOPE=0.0033										
ROUTE	207.02	23	71	.03038	66.41	2.857	1.76351	1.567	3.415	
COMPUTE NM HYD	210.00	-	24	.02613	51.48	2.153	1.54525	1.533	3.078	PER IMP= 47.0
ADD HYD	210.10	24&22	23	.04054	108.40	3.703	1.71298	1.633	4.178	
*S DIVIDE OLF HYD 210.1 BY PROPOSED TOMLINSON PIPE NETWORK										
*S**** OLF HYD 210.13 IS AP #3 *****										
DIVIDE HYD	DIVTOM.2	23	74	.03129	50.00	2.858	1.71298	1.500	2.497	
	210.13	AND	22	.00925	58.40	.845	1.71298	1.633	9.865	
*S ROUTING 210.13 THRU 211 ON TOMLINSON DRIVE TO OUTFALL										

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	PAGE = 3 NOTATION
ROUTE	210.12	22	27	.00925	48.56	.845	1.71354	1.667	8.203	
ADD HYD	DIVTOM.3	74&72	73	.14267	175.00	14.840	1.95024	1.766	1.917	
ADD HYD	207.10	73&71	72	.17305	241.10	17.697	1.91746	1.567	2.177	
*S ROUTE 207.1 THRU 211 IN 60" PIPE										
*S Q(CAP) = 263 CFS, AT SLOPE=0.0118										
ROUTE	211.12	72	73	.17305	240.14	17.697	1.91745	1.567	2.168	
COMPUTE NM HYD	211.00	-	23	.06408	110.04	5.340	1.56261	1.600	2.683	PER IMP= 48.0
*S TOTAL OLF AT TOMLINSON OUTFALL										
*S**** OLF HYD 211.1 IS AP #4 ****										
ADD HYD	211.10	23&27	22	.07333	151.89	6.186	1.58163	1.633	3.236	
*S TOTAL FLOW AT TOMLINSON OUTFALL										
ADD HYD	211.20	22&73	25	.24638	386.68	23.882	1.81750	1.633	2.452	
COMPUTE NM HYD	212.00	-	22	.05122	111.55	5.313	1.94479	1.567	3.403	PER IMP= 74.0
*S DIVIDE SUB-BASIN 212 BETWEEN MH# S-94(OUTFALL) AND MH# S-791										
*S**** OLF HYD 212.03 IS AP #8 *****										
DIVIDE HYD	CB20	22	24	.04540	69.80	4.709	1.94478	1.467	2.402	
	212.03	AND	27	.00582	41.75	.604	1.94478	1.567	11.210	
*S TOTAL FLOW IN THE "EXISTING" JUAN TABO STORM DRAIN SYSTEM										
*S**** PIPE HYD CB20.1 IS AP #8 *****										
ADD HYD	CB20.1	24&29	22	.11338	141.69	11.665	1.92919	1.700	1.953	
*S JUAN TABO ROAD, SOUTH OF CENTRAL										
COMPUTE NM HYD	213.00	-	24	.00828	18.50	1.021	2.31187	1.633	3.490	PER IMP= 100.0
COMPUTE NM HYD	301.00	-	31	.05794	99.24	5.468	1.76955	1.633	2.676	PER IMP= 62.0
*S FROM MH# S-973 TO MH# S-993 IN BEUENA VENTURA STREET										
*S Q(CAP) = 306 CFS ; 60" @ S=0.016										
ROUTE	CB3.02	17	32	.14318	300.07	14.955	1.95848	1.533	3.275	
*S OLF HYD 108.13 THRU 301 ON BEUENA VENTURA STREET										
ROUTE	108.12	18	13	.00155	10.60	.163	1.96180	1.600	10.650	
ADD HYD	301.10	13&31	33	.05949	105.85	5.631	1.77455	1.600	2.780	
*S HYD 301.13 IS TOTAL OLF AT JANE AND BEUENA VENTURA										
*S**** OLF HYD 301.13 IS AP #20 *****										
DIVIDE HYD	CB8	33	34	.05334	70.00	5.048	1.77455	1.533	2.051	
	301.13	AND	35	.00616	35.85	.583	1.77455	1.600	9.099	
ADD HYD	CB8.1	34&32	36	.19651	370.07	20.003	1.90856	1.533	2.942	
*S FROM MH# S-772 TO MH# S-973 IN JANE STREET										
*S Q(CAP) = 356 CFS ; 66" @ S=0.013										
ROUTE	CB8.12	36	31	.19651	369.48	20.003	1.90856	1.567	2.938	
*S OLF HYD 301.13 THRU 302 ON JANE STREET										
ROUTE	301.12	35	32	.00616	27.09	.583	1.77539	1.700	6.876	
COMPUTE NM HYD	302.00	-	33	.02999	51.08	2.830	1.76916	1.633	2.661	PER IMP= 63.0
*S DIVIDE BETWEEN MH# S-772 AND MH# S-973										
DIVIDE HYD	CB9	32	35	.00616	27.09	.583	1.77525	1.700	6.876	
	301.16	AND	36	.00000	.00	.000	.00000	-.033	.000	
*S DIVIDE SUB-BASIN OLF 302 BY MAXIMUM CB INFLOW AT JANE AND CHICO										
*S DIVIDE AT MH# S-772										
DIVIDE HYD	CHICO.CB	33	81	.02118	20.00	1.999	1.76915	1.467	1.475	
	302.03	AND	82	.00881	31.08	.831	1.76915	1.633	5.513	
ADD HYD	CB9.1	31&35	34	.20267	383.78	20.586	1.90451	1.633	2.959	
*S TOTAL PIPE FLOW AT CHICO AND JANE										
ADD HYD	CB9.1	34&81	83	.22385	403.78	22.585	1.89170	1.633	2.818	
*S ROUTING HYD CB9.1 FOR PROPOSED 84" IN JANE STREET NORTH OF CHICO ROAD										
*S Q(CAP) = 398 CFS ; 84" @ S=0.0045 FT/FT ; N=0.014										
ROUTE	CB9.12	83	31	.22385	401.93	22.585	1.89170	1.633	2.805	
*S ROUTING HYD CB9.12 FOR PROPOSED 66" PARALLEL TO FREEWAY										
*S Q(CAP) = 418 CFS ; 66" @ S=0.018										
ROUTE	CHICO.DIV	31	71	.22385	398.79	22.585	1.89170	1.633	2.784	
*S**** OLF HYD 302.1 IS AP #21 *****										
ADD HYD	302.10	82&36	34	.00881	31.08	.831	1.76915	1.633	5.513	
ROUTE	302.12	34	32	.00881	26.48	.831	1.76973	1.733	4.698	
COMPUTE NM HYD	303.00	-	33	.08070	128.69	7.616	1.76955	1.700	2.492	PER IMP= 62.0
ADD HYD	303.10	33&32	34	.08951	154.81	8.447	1.76955	1.700	2.702	
*S DIVIDE BETWEEN MH# S-743 AND MH# S-772 BY CB CAPACITY										
*S**** PIPE HYD CB10 IS AP #22 *****										
DIVIDE HYD	CB10	34	35	.07372	85.00	6.957	1.76955	1.533	1.802	
	303.13	AND	33	.01579	69.81	1.490	1.76955	1.700	6.907	
*S FROM MH# S-704 TO MH# S-743 IN CHICO ROAD										
*S Q(CAP) = 216 CFS										
ROUTE	CB10.12	35	36	.07372	85.00	6.957	1.76956	1.966	1.802	
COMPUTE NM HYD	304.00	-	31	.01342	39.16	1.655	2.31187	1.533	4.559	PER IMP= 100.0
ROUTE	304.03	31	32	.01342	28.02	1.655	2.31189	1.600	3.262	
COMPUTE NM HYD	305.00	-	31	.03919	64.44	3.290	1.57396	1.633	2.569	PER IMP= 49.0

```
                                  FROM  TO                    PEAK        RUNOFF                TIME TO   CFS      PAGE =   4
                       HYDROGRAPH  ID   ID        AREA     DISCHARGE     VOLUME      RUNOFF      PEAK    PER
COMMAND          IDENTIFICATION   NO.  NO.     (SQ MI)      (CFS)       (AC-FT)    (INCHES)   (HOURS)   ACRE     NOTATION

ADD HYD                305.10 31&32  35        .05261       92.16        4.944     1.76216     1.600    2.737
*S TOTAL OLF AT CHICO AND MORRIS
*S****     OLF HYD 305.2 IS AP #22     *****
ADD HYD                305.20 35&33  32        .06840      156.37        6.435     1.76386     1.667    3.572
ROUTE                  305.22   32   33        .06840      127.02        6.435     1.76387     1.766    2.902
COMPUTE NM HYD         306.00    -   31        .03441       83.70        4.243     2.31187     1.600    3.801 PER IMP= 100.0
*S ROUTING SUB-BASIN 306 THRU 308 ( THRU WALL BETWEEN BASIN 306 AND 308)
ROUTE                  306.03   31   32        .03441       74.97        4.243     2.31187     1.667    3.404
*S SUB-BASIN 307 PONDS
COMPUTE NM HYD         307.00    -   34        .02937       64.35        2.258     1.44180     1.500    3.423 PER IMP=  40.0
COMPUTE NM HYD         308.00    -   31        .08051      133.77        6.512     1.51655     1.600    2.596 PER IMP=  45.0
ADD HYD                308.10 31&32  35        .11492      205.71       10.755     1.75468     1.633    2.797
*S EXISTING DETENTION BASIN IN BASIN 308
*S ROUTING RESERVOIR WITH 36" ORIFICE
*S****     PIPE FLOW 308.RES IS AP #23     *****
ROUTE RESERVOIR       308.RES   35   39        .11492       69.84       10.755     1.75468     2.100     .950 AC-FT=    4.46
*S
*S       OUTFLOW(CFS)    STORAGE(AC-FT)    ELEV(FT)
*S            0                0             5456
*S           19               0.09           5458
*S           48               1.25           5460
*S           69               4.09           5462
*S           83              10.36           5464
*S
COMPUTE NM HYD         309.00    -   31        .01291       29.71        1.064     1.54525     1.500    3.596 PER IMP=  47.0
ADD HYD                309.10 33&31  32        .08131      137.75        7.499     1.72914     1.766    2.647
*S DIVIDE BY CB CAPACITY ON CHICO STREET BETWEEN MH# S-704  TO  MH# S-743
*S OLF HYD 309.13 IS AT CHICO AND EUBANK
*S****     OLF HYD 309.13 IS AP #24     *****
DIVIDE HYD               CB11   32   33        .04222       30.00        3.894     1.72914     1.433    1.110
                       309.13  AND   37        .03909      107.75        3.605     1.72914     1.766    4.307
ADD HYD                CB11.1 33&36  38        .11594      115.00       10.851     1.75483     1.966    1.550
*S TOTAL PIPE FLOW AT CHICO AND EUBANK FROM THE CHICO SYSTEM
*S****     PIPE HYD CB11.2 IS AP #24     *****
ADD HYD                CB11.2 38&39  32        .23086      184.59       21.606     1.75476     1.966    1.249
COMPUTE NM HYD         310.00    -   38        .05825      145.16        6.710     2.15990     1.567    3.894 PER IMP=  90.0
COMPUTE NM HYD         311.00    -   39        .10616      208.97       11.041     1.95009     1.633    3.076 PER IMP=  75.0
COMPUTE NM HYD         312.00    -   33        .02461       71.05        2.756     2.09963     1.500    4.511 PER IMP=  85.0
*S BASIN 400 IS SOUTH OF CENTRAL
COMPUTE NM HYD         401.00    -   41        .03371       73.42        3.928     2.18485     1.633    3.403 PER IMP=  90.0
COMPUTE NM HYD         402.00    -   42        .03516       76.91        4.315     2.30112     1.633    3.418 PER IMP=  99.0
*S OLF HYD 103.13 THRU 303 AND 304 ON NORTH SIDE OF CENTRAL STREET
ROUTE                  103.12   15   43        .01204       49.25        1.351     2.10442     1.667    6.390
*S OLF HYD 204.13 THRU 402 ON SOUTH SIDE CENTRAL STREET
ROUTE                  204.12   86   44        .04164      115.32        4.487     2.02019     1.766    4.327
ADD HYD                402.10 42&44  45        .07680      181.32        8.802     2.14877     1.733    3.689
*S DIVIDE BY CB ON NORTH SIDE CENTRAL STREET BETWEEN MH# S-42  AND MH# S-102
*S OLF HYD 103.13 IS FLOW ON CENTRAL'S NORTH SIDE AT ELIZABETH AND CENTRAL                                                  0
*S****     OLF HYD 103.13 IS AP #15     *****
DIVIDE HYD               CB14   43   42        .01168       42.00        1.310     2.10432     1.633    5.620
                       103.13  AND   44        .00037        7.25         .041     2.10432     1.667   30.987
*S DIVIDE BY CB ON SOUTH SIDE CENTRAL STREET BETWEEN MH# S-42  AND MH# S-102
*S OLF HYD 402.13 IS CENTRAL'S TOTAL SOUTH SIDE FLOW AT ELIZABETH AND CENTRAL
*S****     OLF HYD 402.13 IS AP #16     *****
DIVIDE HYD               CB15   45   46        .04224       49.00        4.841     2.14877     1.500    1.812
                       402.13  AND   47        .03456      132.32        3.961     2.14877     1.733    5.982
*S FROM MH# S-42  TO  MH# S-102
*S Q(CAP) = 276 CFS
ROUTE                  CB6.22   99   43        .20518      277.94       22.793     2.08298     1.766    2.117                .0
ADD HYD                CB14.1 42&43  45        .21685      319.72       24.104     2.08412     1.700    2.304                .0
*S TOTAL PIPE FLOW AT CENTRAL AND ELIZABETH                                                                                  .0
*S****     PIPE HYD CB15.1 IS AP #15 AND AP #16     *****                                                                    .0
ADD HYD                CB15.1 45&46  42        .25909      368.72       28.945     2.09466     1.700    2.224
*S TOTAL PIPE FLOW AT ELIZABETH AND COCHITI
ADD HYD               401.1CB 42&41  71        .29280      441.03       32.873     2.10505     1.633    2.353
*S ROUTE PIPE HYD "401.1CB" THRU 403 IN 78" PIPE ON ELIZABETH STREET TO SOUTHERN
*S Q(CAP) = 430 CFS, AT N=0.014 AND SLOPE=0.0072
ROUTE                401.12CB   71   72        .29280      437.08       32.873     2.10505     1.667    2.332
*S OLF HYD 103.13 THRU 304 ON NORTH SIDE CENTRAL STREET; BETWEEN EUBANK AND ELIZ
ROUTE                  103.12   44   46        .00037        1.17         .041     2.11845     1.700    5.023               11
*S OLF HYD 402.13 THRU 402 ON SOUTH SIDE CENTRAL STREET; BETWEEN EUBANK AND ELIZ
ROUTE                  402.12   47   42        .03456      109.12        3.961     2.14892     1.833    4.933
```

```
                                  FROM   TO                    PEAK        RUNOFF                 TIME TO    CFS      PAGE =   5
                     HYDROGRAPH   ID     ID        AREA      DISCHARGE     VOLUME      RUNOFF       PEAK      PER
COMMAND            IDENTIFICATION NO.    NO.      (SQ MI)     (CFS)       (AC-FT)    (INCHES)    (HOURS)    ACRE      NOTATION

COMPUTE NM HYD        403.00      -      41       .05744      110.01       5.677     1.85327      1.600     2.992 PER IMP=  70.0
COMPUTE NM HYD        404.00      -      44       .04882       89.80       4.825     1.85327      1.633     2.874 PER IMP=  70.0
*S **** SUB-BASIN 405B ****
*S ** EASTERN HALF OF OLD BASIN 405 ****
COMPUTE NM HYD          405B      -      90       .02687       74.78       3.107     2.16836      1.533     4.349 PER IMP=  90.0
*S ROUTE 405B THRU BASIN 404 IN FUTURE DRAINAGE CHANNEL ***
*S FUTURE DRAINAGE CHANNEL TO BE LOCATED ON NORTH EDGE OF BASIN 404 ***
ROUTE                 R.405B    90     91       .02687       74.45       3.107     2.16837      1.533     4.330
*S  ADD SUB-BASIN 404 TO R.405B - NEW AP-29B ***
ADD HYD               AP.29B 91&44     92       .07569      153.44       7.933     1.96511      1.567     3.168
*S ROUTE AP.29B THROUGH PROPOSED 404 POND AT NW CORNER OF BASIN 404
ROUTE RESERVOIR      404POND    92     44       .07569       86.75       7.933     1.96512      1.833     1.791 AC-FT=    2.61
*S DIVIDE TOTAL FLOW AT NORTHWEST CORNER OF BASIN 404 INTO STORM DRAIN FLOW *
*S AND OVERFLOW THAT SPILLS INTO EUBANK BOULEVARD AT BELL *
DIVIDE HYD          AP29B.SD    44     90       .07295       75.00       7.645     1.96510      1.667     1.606
                   AP29B.OLF   AND     91       .00274       11.75        .287     1.96511      1.833     6.702
COMPUTE NM HYD        506.00      -      56       .01509       32.37       1.549     1.92516      1.567     3.352 PER IMP=  75.0
*S ROUTING 506 THRU 404 ON EUBANK BLVD TO AP-29A
ROUTE                 AP.29A    56     58       .01509       29.83       1.549     1.92516      1.633     3.089
*S DIVIDE FLOW FROM SOUTHERN BLVD (ROUTED TO BELL) INTO SD FLOW *
*S AND OVERFLOW THAT CONTINUES NORTH ON EUBANK BLVD *
DIVIDE HYD          AP29A.SD    58     44       .01509       29.83       1.549     1.92512      1.633     3.089
                   AP29A.OLF   AND     92       .00000         .00        .000      .00000      -.033      .000
*S  ADD AP29A.OLF TO AP29B.OLF TO OBTAIN TOTAL STREET FLOW AT EUB/BELL***
ADD HYD            AP29C.OLF 92&91     56       .00274       11.75        .287     1.96511      1.833     6.702
ROUTE              ACOMA.OUT   56     91       .00274       11.51        .287     1.96699      1.866     6.565
*S  ADD AP29A.SD TO AP29B.SD TO OBTAIN TOTAL SD FLOW AT EUBANK/BELL ***
ADD HYD             AP29C.SD 44&90     96       .08804      104.08       9.195     1.95825      1.667     1.847
*S ROUTING STORM DRAIN FLOW (AP29C.SD) AT EUBANK/BELL
*S NORTH ALONG EUBANK BLVD TO CONFLUENCE WITH HOME DEPOT SD (MH-1942)
ROUTE               R.APG.SD    96     92       .08804      103.18       9.195     1.95826      1.700     1.831
COMPUTE NM HYD        406.00      -      44       .01810       45.54       2.173     2.25016      1.567     3.930 PER IMP=  95.0
ADD HYD               406.10 42&44     45       .05266      130.71       6.133     2.18369      1.833     3.878
*S DIVIDE BY CB ON NORTH SIDE CENTRAL STREET BETWEEN MH# S-4  AND MH# S-42
*S****     OLF HYD 103.23 IS AP #18     *****
DIVIDE HYD              CB16    46     42       .00037        1.17        .041     2.11458      1.700     5.023
                      103.23   AND     44       .00000         .00        .000      .00000      -.033      .000
*S DIVIDE BY CB ON SOUTH SIDE CENTRAL STREET BETWEEN MH# S-4  AND  MH# S-42
*S****     OLF HYD 406.13 IS AP #17     *****
DIVIDE HYD              CB17    45     48       .03265       49.00       3.802     2.18369      1.567     2.345
                      406.13   AND     49       .02002       81.71       2.331     2.18369      1.833     6.379
*S TOTAL PIPE FLOW AT CENTRAL AND EUBANK FROM EXISTING 66"
*S****     PIPE HYD CB17.1 IS AP #17     *****
ADD HYD               CB17.1 42&48     40       .03301       50.17       3.843     2.18293      1.700     2.375
*S DIVIDE PIPE HYD CB17.1 BY MAXIMUM PIPE CAPACITY OF 60" IN EUBANK
DIVIDE HYD           CB17.13    40     43       .03301       50.17       3.843     2.18293      1.700     2.375
                    CB17.OLF   AND     48       .00000         .00        .000      .00000      -.033      .000
COMPUTE NM HYD        501.00      -      51       .01967       41.55       2.275     2.16836      1.633     3.301 PER IMP=  90.0
*S ROUTING 213 THRU 501 ON SOUTHERN STREET
ROUTE                 213.02    24     52       .00828       12.54       1.021     2.31188      1.800     2.366
ADD HYD               403.10 41&52     53       .06572      118.31       6.698     1.91102      1.633     2.813
*S TOTAL OLF AT SOUTHERN AND ELIZABETH
ADD HYD               501.10 51&53     52       .08539      159.86       8.973     1.97030      1.633     2.925
*S ADD PIPE HYD 401.12CB WITH OLF HYD 501.1 FOR PROPOSED PIPE
ADD HYD               501.20 72&52     71       .37819      592.24      41.846     2.07462      1.667     2.447
*S ROUTING 501.2 THRU 502 ON ELIZABETH STREET WITH PROPOSED 90" PIPE
*S Q(CAP) = 584 CFS, AT SLOPE=0.0067
ROUTE                 501.22    71     73       .37819      587.82      41.846     2.07462      1.667     2.429
COMPUTE NM HYD        502.00      -      52       .19070      313.77      16.069     1.57996      1.633     2.571 PER IMP=  49.0
COMPUTE NM HYD        503.00      -      55       .18018      290.84      15.045     1.56562      1.633     2.522 PER IMP=  48.0
COMPUTE NM HYD        504.00      -      56       .00886       15.57        .482     1.02027      1.500     2.746 PER IMP=  10.0
COMPUTE NM HYD        505.00      -      59       .06230       72.84       3.390     1.02027      1.633     1.827 PER IMP=  10.0
*S TOTAL PROPOSED PIPE FLOW AT END OF SUB-BASIN 502
ADD HYD               502.10 52&73     72       .56889      893.16      57.915     1.90881      1.667     2.453
ADD HYD               504.10 56&55     53       .18904      301.08      15.527     1.54005      1.633     2.489
ADD HYD               505.10 53&59     52       .25134      373.91      18.917     1.41121      1.633     2.324
*S TOTAL FLOW TO PROPOSED DETENTION BASIN
ADD HYD               505.20 52&72     73       .82023     1264.54      76.832     1.75633      1.633     2.409
*S ROUTING PROPOSED RESERVOIR WITH 66" OUTLET
ROUTE RESERVOIR      504.RES    73     71       .82023      291.91      76.832     1.75633      2.333      .556 AC-FT=   43.11
*S
*S          OUTFLOW(CFS)    STORAGE(AC-FT)    ELEV(FT)
```

```
                                FROM  TO                    PEAK        RUNOFF                TIME TO   CFS      PAGE =   6
                   HYDROGRAPH   ID    ID        AREA     DISCHARGE     VOLUME      RUNOFF      PEAK    PER
COMMAND          IDENTIFICATION NO.  NO.      (SQ MI)     (CFS)       (AC-FT)    (INCHES)   (HOURS)   ACRE      NOTATION

*S                50              0.081        76
*S                88              3.575        78
*S               170             12.981        80
*S               240             27.687        82
*S               282             36.367        83
*S               295             45.216        84
*S               310             54.233        85
*S
*S ROUTING 504.RES IN PROPOSED ELIZABETH STREET'S 78" THRU BASIN 509/510
*S Q(CAP) = 472 CFS ; SLOPE=0.0094
ROUTE                     504.R2  71   72        .82023      291.83      76.832     1.75633    2.366     .556
*S *** SUB-BASIN 405AA ****
*S *** HOME DEPOT DRAINAGE BASIN "A" *
COMPUTE NM HYD            405AA   -    94        .00799       23.65        .924     2.16836    1.500    4.623 PER IMP=  90.0
*S ** ROUTE 405AA THROUGH HOME DEPOT PARKING LOT POND
*S ** USING A 4" ORIFICE PLATE
ROUTE RESERVOIR           PONDA   94   97        .00799         .77        .895     2.09947    2.566     .150 AC-FT=     .77
*S * SUB-BASIN 405AB ****
*S HOME DEPOT DRAINAGE BASIN "B" *
COMPUTE NM HYD            405AB   -    98        .01757       51.96       2.032     2.16836    1.500    4.621 PER IMP=  90.0
*S  ADD SUB-BASIN 405AB TO OUTFLOW FROM PONDA ROUTING
ADD HYD                  BPONDA 98&97  99        .02556       52.51       2.927     2.14681    1.500    3.210
*S ROUTE BPONDA THROUGH HOME DEPOT PARKING LOT POND B
ROUTE RESERVOIR          AP.29D  99   97        .02556        3.02       2.910     2.13464    2.433     .185 AC-FT=    1.64
*S  ADD HOME DEPOT FLOW TO STORM DRAIN FLOW AT R.AP.G.SD
ADD HYD                    AP.D 97&92  94        .11360      105.91      12.105     1.99794    1.700    1.457
ROUTE                    AP.29E  94   97        .11360      105.97      12.104     1.99772    1.700    1.458
COMPUTE NM HYD           507.00   -   51        .02184       66.23       2.609     2.24012    1.500    4.738 PER IMP=  95.0
COMPUTE NM HYD           508.00   -   52        .02059       53.72       2.035     1.85327    1.500    4.077 PER IMP=  70.0
*S TOTAL FLOW AT EUBANK AND 500' SOUTH OF SOUTHERN
*S****     OLF HYD 508.1 IS AP #30     *****
ADD HYD                  508.10 51&52  57        .04243      119.95       4.644     2.05238    1.500    4.417
COMPUTE NM HYD           509.00   -   51        .02582       67.36       2.552     1.85327    1.500    4.076 PER IMP=  70.0
COMPUTE NM HYD           510.00   -   52        .07573      167.53       7.485     1.85328    1.567    3.457 PER IMP=  70.0
ADD HYD                  510.10 52&51  54        .10155      230.14      10.037     1.85327    1.533    3.541
*S TOTAL PROPOSED PIPE FLOW AT ELIZABETH AND EUBANK
ADD HYD                  510.20 72&54  71        .92178      345.85      86.869     1.76701    2.000     .586
*S ROUTING 510.1 IN PROPOSED 96" FROM ELIZABTH TO SUB-BASIN 606 ON EUBANK
*S Q(CAP) = 465 CFS AT SLOPE=0.003
ROUTE                    510.22  71   72        .92178      344.01      86.869     1.76701    2.033     .583
*S  ADD PIPE HYD AP.29E TO PIPE HYD CB17.13 ***
*S  TOTAL PROPOSED PIPE FLOW AT EUBANK AND CENTRAL
*S****     PIPE HYD EUB.1 IS AP #19     *****
ADD HYD                    EUB.1 97&43  81        .14661      156.14      15.947     2.03943    1.700    1.664
*S FROM MH# S-704  TO  MH# S-4
*S Q(CAP) = 187 CFS
ROUTE                   CB17.12  81   42        .14661      153.08      15.943     2.03889    1.733    1.631
*S TOTAL PIPE FLOW AT EUBANK AND CHICO
ADD HYD                  CB17.2 32&42  43        .37747      332.60      37.548     1.86512    1.800    1.377
*S DIVIDE PIPE HYD CB17.2 BY MAXIMUM PIPE CAPACITY OF 60" IN EUBANK
*S OLF HYD CB17.OLF2 IS AMOUNT OF FLOW THAT COMES OUT FROM MH S-704
*S****     PIPE HYD CB17.23 IS AP #25     *****
DIVIDE HYD              CB17.23  43   42        .33611      235.00      33.434     1.86512    1.567    1.092
                      CB17.OLF2  AND  44        .04136       97.60       4.114     1.86512    1.800    3.687
*S FROM MH# S-503  TO  MH# S-704
*S Q(CAP) = 235 CFS
ROUTE                   CB17.22  42   43        .33611      235.00      33.432     1.86499    1.833    1.092
*S TOTAL OLF FLOW AT EUBANK AND CHICO
ADD HYD                  309.20 37&44  46        .08045      204.98       7.719     1.79905    1.766    3.981
*S OLF HYD 309.13 THRU 310 ON EUBANK STREET; FROM CHICO TO COPPER
ROUTE                    309.22  46   42        .08045      198.34       7.719     1.79911    1.833    3.852
ADD HYD                  310.10 42&38  48        .13870      268.15      14.429     1.95063    1.800    3.021
*S DIVIDE BY CB ON EUBANK STREET BETWEEN MH# S-704  AND  MH# S-503, INCLUDING 48
*S****     OLF HYD 310.13 IS AP #26     *****
DIVIDE HYD                 CB12  48   44        .11413      160.00      11.873     1.95063    1.567    2.191
                         310.13  AND  49        .02457      108.15       2.556     1.95063    1.800    6.878
*S TOTAL PIPE FLOW AT EUBANK AND COPPER
*S****     PIPE HYD CB17.3 IS AP #27     *****
ADD HYD                  CB17.3 44&43  48        .45024      395.00      45.305     1.88669    1.833    1.371
COMPUTE NM HYD           601.00   -   61        .03286       75.65       3.322     1.89569    1.533    3.597 PER IMP=  71.0
*S SUB-BASIN 602 EITHER PONDS OR GOES INTO TIJERAS ARROYO
```

COMMAND	HYDROGRAPH IDENTIFICATION	FROM ID NO.	TO ID NO.	AREA (SQ MI)	PEAK DISCHARGE (CFS)	RUNOFF VOLUME (AC-FT)	RUNOFF (INCHES)	TIME TO PEAK (HOURS)	CFS PER ACRE	PAGE = 7 NOTATION
COMPUTE NM HYD	602.00	-	62	.01436	26.02	.770	1.00588	1.500	2.832	PER IMP= .0
COMPUTE NM HYD	603.00	-	63	.01970	35.70	1.057	1.00588	1.500	2.831	PER IMP= .0
COMPUTE NM HYD	604.00	-	64	.06839	127.07	6.348	1.74050	1.600	2.903	PER IMP= 60.5
*S ROUTING 604 THRU 603 ON GRAVEL ROAD										
ROUTE	604.02	64	65	.06839	123.73	6.348	1.74050	1.633	2.827	
*S OLF HYD 604.1 EITHER PONDS OR GOES INTO TIJERAS ARROYO										
*S**** OLF HYD 604.1 IS AP #33 *****										
ADD HYD	604.10	63&65	64	.08809	147.32	7.405	1.57621	1.633	2.613	
COMPUTE NM HYD	605.00	-	65	.04536	80.92	5.286	2.18485	1.733	2.787	PER IMP= 90.0
COMPUTE NM HYD	606.00	-	66	.06633	154.15	7.729	2.18485	1.600	3.631	PER IMP= 90.0
*S OLF HYD 606.1 GOES INTO EXISTING RETENTION BASIN										
ADD HYD	606.10	65&66	67	.11169	217.95	13.015	2.18484	1.633	3.049	
ADD HYD	606.20	72&67	73	1.03347	542.01	99.884	1.81217	1.633	.819	
*S ROUTING 606.2 IN PROPOSED 96" THRU SUB-BASIN 607										
*S Q(CAP) = 600 CFS AT SLOPE=0.005										
ROUTE	606.22	73	71	1.03347	531.99	99.884	1.81217	1.700	.804	
COMPUTE NM HYD	607.00	-	65	.12156	223.97	12.545	1.93498	1.667	2.879	PER IMP= 68.0
ADD HYD	607.10	65&71	72	1.15503	752.95	112.429	1.82509	1.667	1.019	
*S ROUTING 607.1 IN PROPOSED 96" ; S=0.0082 FT/FT										
ROUTE	607.12	72	71	1.15503	745.74	112.429	1.82509	1.700	1.009	
COMPUTE NM HYD	608.00	-	65	.08609	74.20	4.330	.94295	1.700	1.347	PER IMP= 15.0
*S TOTAL FLOW TO PROPOSED OUTFALL CHANNEL										
ADD HYD	608.10	65&71	68	1.24112	819.94	116.758	1.76390	1.700	1.032	
*S**** SUB-BASIN 609 IS AP #35 *****										
COMPUTE NM HYD	609.00	-	65	.02603	25.38	1.062	.76501	1.600	1.523	PER IMP= .0
FINISH										

ENGINEER'S ESTIMATE

COMPARISON OF STORM DRAIN OPTIONS
EUBANK BOULEVARD IMPROVEMENTS

			OPTION							
			1 (48" - WEST SIDE)		2 (36" - WEST SIDE)		3 (48" - EAST SIDE)		4 (36" - EAST SIDE)	
ITEM	UNIT	UNIT PRICE	QTY	COST	QTY	COST	QTY	COST	QTY	COST
TRENCH, BACKFILL, 18"-36",8'-12'	LF	$16.75	0	$0.00	350	$5,862.50	0	$0.00	345	$5,778.75
TRENCH, BACKFILL, 18"-36",12'-16'	LF	$24.00	0	$0.00	420	$10,080.00	0	$0.00	460	$11,040.00
TRENCH, BACKFILL, 42"-60",8'-12'	LF	$27.75	650	$18,037.50	300	$8,325.00	645	$17,898.75	300	$8,325.00
TRENCH, BACKFILL, 42"-60",12'-16'	LF	$43.25	580	$25,085.00	160	$6,920.00	560	$24,220.00	100	$4,325.00
36" RCP, III	LF	$47.00	0	$0.00	770	$36,190.00	0	$0.00	805	$37,835.00
48" RCP, III	LF	$71.00	1230	$87,330.00	460	$32,660.00	1205	$85,555.00	400	$28,400.00
MANHOLE, 5" DIA, 6'-10' DEEP	EA	$2,440.00	0	$0.00	2	$4,880.00	0	$0.00	2	$4,880.00
MANHOLE, 5" DIA, 10'-14' DEEP	EA	$2,660.00	0	$0.00	1	$2,660.00	0	$0.00	1	$2,660.00
MANHOLE, 6" DIA, 6'-10' DEEP	EA	$2,920.00	3	$8,760.00	1	$2,920.00	3	$8,760.00	1	$2,920.00
MANHOLE, 6" DIA, 10'-14' DEEP	EA	$3,425.00	3	$10,275.00	2	$6,850.00	3	$10,275.00	2	$6,850.00
EXISTING ASPHALT PAVEMENT, >4", REMOVE & DISPOSE	SY	$3.50	850	$2,975.00	850	$2,975.00	975	$3,412.50	975	$3,412.50
ARTERIAL ASPHALT PAVING, 6"	SY	$16.00	790	$12,640.00	790	$12,640.00	0	$0.00	0	$0.00
ARTERIAL ASPHALT PAVING, 12"	SY	$29.50	60	$1,770.00	60	$1,770.00	975	$28,762.50	975	$28,762.50
4" ASPHALT CONCRETE BASE, SD 2465	SY	$8.00	850	$6,800.00	850	$6,800.00	975	$7,800.00	975	$7,800.00
24" WATERLINE CROSSING	LS	$20,000.00	1	$20,000.00	1	$20,000.00	0	$0.00	0	$0.00
SPECIAL UTILITY CROSSINGS:										
60" x 38" ELLIPTICAL PIPE	LF	$100.00	40	$4,000.00	0	$0.00	0	$0.00	0	$0.00
45" x 29" ELLIPTICAL PIPE	LF	$70.00	0	$0.00	40	$2,800.00	0	$0.00	0	$0.00
MANHOLE, 5" DIA, 10'-14' DEEP	EA	$2,660.00	0	$0.00	0	$0.00	0	$0.00	1	$2,660.00
MANHOLE, 6" DIA, 10'-14' DEEP	EA	$3,425.00	0	$0.00	2	$6,850.00	1	$3,425.00	0	$0.00
MANHOLE, 8" DIA, 10'-14' DEEP	EA	$6,200.00	2	$12,400.00	0	$0.00	0	$0.00	0	$0.00
WATERLINE CROSSING	LS	$3,000.00	1	$3,000.00	0	$0.00	1	$3,000.00	1	$3,000.00
TOTAL				**$213,072.50**		**$171,182.50**		**$193,108.75**		**$158,648.75**

ENGINEER'S ESTIMATE

COMPARISON OF STORM DRAIN OPTIONS
EUBANK BOULEVARD IMPROVEMENTS

			OPTION							
			1 (48" - WEST SIDE)		2 (36" - WEST SIDE)		3 (48" - EAST SIDE)		4 (36" - EAST SIDE)	
ITEM	UNIT	UNIT PRICE	QTY	COST	QTY	COST	QTY	COST	QTY	COST
TRENCH, BACKFILL, 18"-36",8'-12'	LF	$16.75	0	$0.00	350	$5,862.50	0	$0.00	345	$5,778.75
TRENCH, BACKFILL, 18"-36",12'-16'	LF	$24.00	0	$0.00	420	$10,080.00	0	$0.00	460	$11,040.00
TRENCH, BACKFILL, 42"-60",8'-12'	LF	$27.75	650	$18,037.50	300	$8,325.00	645	$17,898.75	300	$8,325.00
TRENCH, BACKFILL, 42"-60",12'-16'	LF	$43.25	580	$25,085.00	160	$6,920.00	560	$24,220.00	100	$4,325.00
36" RCP, III	LF	$47.00	0	$0.00	770	$36,190.00	0	$0.00	805	$37,835.00
48" RCP, III	LF	$71.00	1230	$87,330.00	460	$32,660.00	1205	$85,555.00	400	$28,400.00
MANHOLE, 5" DIA, 6'-10' DEEP	EA	$2,440.00	0	$0.00	2	$4,880.00	0	$0.00	2	$4,880.00
MANHOLE, 5" DIA, 10'-14' DEEP	EA	$2,660.00	0	$0.00	1	$2,660.00	0	$0.00	1	$2,660.00
MANHOLE, 6" DIA, 6'-10' DEEP	EA	$2,920.00	3	$8,760.00	1	$2,920.00	3	$8,760.00	1	$2,920.00
MANHOLE, 6" DIA, 10'-14' DEEP	EA	$3,425.00	3	$10,275.00	2	$6,850.00	3	$10,275.00	2	$6,850.00
EXISTING ASPHALT PAVEMENT, >4", REMOVE & DISPOSE	SY	$3.50	850	$2,975.00	850	$2,975.00	975	$3,412.50	975	$3,412.50
ARTERIAL ASPHALT PAVING, 6"	SY	$16.00	790	$12,640.00	790	$12,640.00	0	$0.00	0	$0.00
ARTERIAL ASPHALT PAVING, 12"	SY	$29.50	60	$1,770.00	60	$1,770.00	975	$28,762.50	975	$28,762.50
4" ASPHALT CONCRETE BASE, SD 2465	SY	$8.00	850	$6,800.00	850	$6,800.00	975	$7,800.00	975	$7,800.00
24" WATERLINE CROSSING	LS	$20,000.00	1	$20,000.00	1	$20,000.00	0	$0.00	0	$0.00
SPECIAL UTILITY CROSSINGS:										
60" x 38" ELLIPTICAL PIPE	LF	$100.00	40	$4,000.00	0	$0.00	0	$0.00	0	$0.00
45" x 29" ELLIPTICAL PIPE	LF	$70.00	0	$0.00	40	$2,800.00	0	$0.00	0	$0.00
MANHOLE, 5" DIA, 10'-14' DEEP	EA	$2,660.00	0	$0.00	0	$0.00	0	$0.00	1	$2,660.00
MANHOLE, 6" DIA, 10'-14' DEEP	EA	$3,425.00	0	$0.00	2	$6,850.00	1	$3,425.00	0	$0.00
MANHOLE, 8" DIA, 10'-14' DEEP	EA	$6,200.00	2	$12,400.00	0	$0.00	0	$0.00	0	$0.00
WATERLINE CROSSING	LS	$3,000.00	1	$3,000.00	0	$0.00	1	$3,000.00	1	$3,000.00
TOTAL				**$213,072.50**		**$171,182.50**		**$193,108.75**		**$158,648.75**

DRAINAGE INFORMATION SHEET
(REV. 11/01/2001)

L-21/D54

PROJECT TITLE: Eubank Boulevard Improvements, Drainage Report ZONE ATLAS/DRNG. FILE # ~~L-20~~

DRB #: _____ EPC #: _____ WORK ORDER #: _____

LEGAL DESCRIPTION: _____

CITY ADDRESS: _____

ENGINEERING FIRM: BOHANNAN-HUSTON INC. CONTACT: Craig W. Hoover, P.E.
ADDRESS: 7500 JEFFERSON NE, ALB. NM 87109 PHONE: (505) 823-1000
CITY, STATE: ALBUQUERQUE, NM ZIP CODE: 87109
OWNER: CITY OF ALBUQUERQUE CONTACT: BRAD BINGHAM, P.E.
ADDRESS: PO BOX 1293 PHONE: (505) 924-3986
CITY, STATE: ALBUQUERQUE, NM ZIP CODE; 87103
ARCHITECT: _____ CONTACT: _____
ADDRESS: _____ PHONE: _____
CITY, STATE: _____ ZIP CODE; _____
SURVEYOR: _____ CONTACT: _____
ADDRESS: _____ PHONE: _____
CITY, STATE: _____ ZIP CODE; _____
CONTRACTOR: _____ CONTACT: _____
ADDRESS: _____ PHONE: _____
CITY, STATE: _____ ZIP CODE; _____

CHECK TYPE OF SUBMITTAL:

X DRAINAGE REPORT
___ DRAINAGE PLAN
___ CONCEPTUAL GRADING & DRAINAGE PLAN
___ GRADING PLAN
___ EROSION CONTROL PLAN
___ ENGINEER"S CERTIFICATION
___ CLMOR/LOMAR
___ TRAFFIC CIRCULATION LAYOUT (TLC)
___ ENGINEERS CERTIFICATION (TLC)
___ ENGINEERS CERTIFICATION (DRB APPR.SITE PLAN)
___ OTHER

CHECK TYPE OF APPROVAL SOUGHT:

___ SIA / FINANCIAL GUARANTEE RELEASE
___ PRELIMINARY PLAT APPROVAL
___ S. DEV. PLAN FOR SUB'D. APPROVAL
___ S. DEV. PLAN FOR BLDG. PERMIT APPROVAL
___ SECTOR PLAN APPROVAL
___ FINAL PLAT APPROVAL
___ FOUNDATION PERMIT APPROVAL
___ BUILDING PERMIT APPROVAL
___ CERTIFICATE OF OCCUPANCY APPROVAL (PERM)
___ CERTIFICATE OF OCCUPANCY APPROVAL (TEMP)
___ GRADING PERMIT APPROVAL
___ PAVING PERMIT APPROVAL
___ WORK ORDER APPROVAL
___ OTHER (SPECIFY)

WAS A PRE-DESIGN CONFERENCE ATTENDED:
___ YES
___ NO
___ COPY PROVIDED

RECEIVED
JAN 11 2002
HYDROLOGY SECTION

DATE SUBMITTED: January 11, 2002 BY: CRAIG W. HOOVER, P.E.

Requests for approval of Site Development Plans and /or Subdivision Plats shall be accompanied by a drainage submittal. The particular nature, location and scope of the proposed development defines the degree of drainage detail. One or more of the following levels of submittal may be required based on the following:

1. Conceptual Grading and Drainage Plan: Required for approval of Site Development Plans greater than five (5) acres and Sector Plans.
2. Drainage Plans: Required for building permits, grading permits, paving permits and site plans less than five (5) acres.
3. Drainage Report: Required for subdivisions containing more than ten (10) lots or constituting five (5) acres or more.

City of Albuquerque
P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

January 28, 2002

Craig Hoover, PE
Bohannan Huston, Inc.
7500 Jefferson NE
Albuquerque, NM 87109

Re: Eubank Blvd Improvements Drainage Report
Engineer's Stamp not dated (L21/D54)

Dear Mr. Hoover,

Based upon the information provided in your submittal dated 1-11-02, the above referenced report cannot be approved for Work Order requirements until the following minor comments are addressed.

- The basins for this project should include all of the R/W, not just up to the sidewalk. Please include a typical section in your resubmittal in order to verify land treatments.

- There has been discussion of reducing the retention pond by keeping the APS site runoff on this lot. If the property owner agrees, this is acceptable since you are installing the storm drain large enough to accept developed runoff per the Manzano Mesa masterplan (55 cfs). Please address this in your next submittal.

- Please amend your statement about the floodplain on the APS property to say, "storm drain improvements...will allow APS to apply for a map amendment from FEMA".

- Please denote on the street flow calculation sheets where the one lane dry depth would be and add corresponding denotation on the sump calculation sheet. It would also be helpful to mark up the inlet nomograph as well.

Please stamp and date your next submittal. If you have any questions, you can contact me at 924-3986.

Sincerely,

Bradley L. Bingham

Bradley L. Bingham, PE
Sr. Engineer, PWD
Development and Building Services

C: Annette Montoya, CoA
file

THE CITY OF ALBUQUERQUE IS AN EQUAL OPPORTUNITY/REASONABLE ACCOMMODATION EMPLOYER

EUBANK BOULEVARD IMPROVEMENTS, KIRTLAND AIR FORCE BASE TO CENTRAL AVENUE DRAINAGE REPORT

January 9, 2001

PREPARED FOR:

CITY OF ALBUQUERQUE
P.O. BOX 1293
ALBUQUERQUE, NM 87103

* * * * *

* * * * *

TABLE OF CONTENTS

I. INTRODUCTION 1
II. METHODOLOGIES AND REFERENCES 1
III. EXISTING HYDRAULIC/HYDROLOGIC CONDITIONS 1
IV. STORM DRAIN DESIGN 2
A. Hydrologic Analysis and Results 2
B. Proposed Drainage Improvements 3
C. Hydraulic Analysis of Proposed Systems 8
V. CONCLUSION 9

TABLES

TABLE 1 3
TABLE 2 4

APPENDICES

APPENDIX A - Excerpts from the "City of Albuquerque Development Process Manual"

APPENDIX B - Plates

PLATE 1 – Drainage Basin Map

PLATE 2 – Drainage Basin Map

APPENDIX C - Calculations

Drainage Basin Hydrology Calculations

Typical Street Section

Street Section Capacity Calculations

Sump Inlet (Type A and Type D) Capacity Calculations

On-Grade Inlet Capacity Nomograph

APPENDIX D - Hydraulic Grade Line Profiles

APPENDIX E - Eubank Blvd. 54-inch Storm Drain As-Builts

APPENDIX F - Correspondence

I. INTRODUCTION

The purpose of this report is to address the drainage improvements for Eubank Boulevard from Kirtland Air Force Base to Central Avenue. Included in this report:

- data supporting the proposed Eubank Blvd. storm drain improvements.
- data supporting the proposed size and location of the temporary retention pond.

The basis for this report is the approved "Manzano Mesa Master Drainage Study", Smith Engineering Company, August 1996.

II. METHODOLOGIES AND REFERENCES

This report is prepared in Accordance with City of Albuquerque Development Process Manual (DPM) criteria; specifically, the procedures outlined in DPM Section 22.2. This report also references the approved "Manzano Mesa Master Drainage Study," Smith Engineering Company, August 1996.

III. EXISTING HYDRAULIC/HYDROLOGIC CONDITIONS

The existing hydraulic conditions along the Eubank Blvd. Improvement Project vary greatly from the northern section of the project to the southern. The property adjacent to Eubank Blvd. from Southern Boulevard to Central Avenue is currently developed and serviced by an existing storm drain system that conveys flows north toward Central Avenue. Curb and gutter exists along this portion of Eubank Blvd. and conveys the runoff to catch basins connected to the storm drain.

Eubank Blvd. from Southern Boulevard south to Kirtland AFB is mainly undeveloped with scattered areas of development existing along the east side of the roadway. There is currently no curb and gutter along this section of roadway except for approximately 1400 feet of curb and gutter located from Opportunity Avenue southward. A storm drain parallels Eubank Blvd. from the future Innovation Parkway (Stephen Moody Street) and continues south, past Research Road. This storm drain, designated as System "A" in the Manzano Mesa Master Drainage Plan, currently receives

flow from developments in the area and will also ultimately drain a portion of Central Avenue east of Elizabeth Street.

A FEMA Floodplain of approximately 2.2 acres exists on two parcels of land adjacent to Eubank Blvd. The land containing the northern portion of the floodplain is owned by the Albuquerque Public Schools (APS) and the southern section of the floodplain is within land belonging to GSL Properties of Portland Oregon. In the near future, the southern parcel of land will be the site of the Manzano Mesa Apartments. The proposed drainage improvements for the apartment complex will remove the floodplain located on the southern parcel of land. The storm drain improvements proposed in this drainage report will ~~remove the floodplain within the APS property~~. allow APS to apply for a map amendment.

IV. STORM DRAIN DESIGN

A. Hydrologic Analysis and Results

The hydrology for the fully developed conditions model was based on the City of Albuquerque Development Process Manual, Section 22.2, Hydrology (DPM). The 10-year storm was the basis for analyzing the street hydraulics and the proposed storm drain was sized to accommodate the100-year storm. The procedure described in Part A of the DPM was used for the analysis, and excerpts of this procedure are located in Appendix A. This procedure is an accepted method for calculating runoff volumes and peak flow rates from drainage basins that are 40 acres or smaller.

The procedure is based on the Rational Method and initial abstraction/uniform infiltration precipitation losses and utilizes four different land treatments. The land treatments range from undisturbed land (Type A) to impervious surfaces (Type D). Table 1 summarizes the land treatment types utilized for each contributing drainage basin in the fully developed conditions model. Each of the street drainage basins, with the exception of the APS property and basin 1, contains minimal areas considered to be Land Treatment Type C (median

landscaping), but this area was determined to be negligible, and the street basins were therefore modeled as being 100% Land Treatment Type D.

This should be done for full section –

TABLE 1

LAND TREATMENT TYPES AND BASIN FLOWS - FULLY DEVELOPED CONDITIONS

DRAINAGE BASIN	TOTAL AREA (ACRES)	LAND TREATMENT TYPES (%)				PEAK FLOW (CFS)	RUNOFF VOLUME (CF)
		A	B	C	D		
1	2.53	0	0	10	90	17.7	
2	1.82	0	0	0	100	9.1	
3	3.15	0	0	0	100	14.8	
4	3.21	0	0	0	100	16.1	
5	2.84	0	0	0	100	14.3	48040
6	2.66	0	0	0	100	13.4	45000
7	0.61	0	0	0	100	3.1	
8	0.43	0	0	0	100	2.2	
9	1.38	0	0	0	100	6.9	
10	2.3	0	0	0	100	11.5	
11	1.18	0	0	0	100	6.0	
APS Property	13.30	0	25	25	50	55.0	
APS Property*	13.30	85	0	15	0	28.0	36450

*Existing conditions assumed for this basin only.

This method also takes into account the Precipitation Zone that the project lies within. The entire Eubank Blvd. project lies within Precipitation Zone 3 as defined by the DPM.

Plates 1 and 2 in Appendix C contain the fully developed conditions basin maps.

B. Proposed Drainage Improvements

The proposed Eubank Blvd. storm drain improvements were divided into 4 systems based on the locations of high points along Eubank Blvd. Table 2 summarizes the basins

drained by each of the systems. Plates 1 and 2 display the limits of each system and related drainage basins.

TABLE 2

DRAINAGE SYSTEMS

DRAINAGE SYSTEM	BASINS DRAINED BY EACH SYSTEM
1	1, 2
2	3, 4
3	5, 6, APS
4	7, 8, 9, 10, 11

A cross section of the street within each of the system limits was analyzed to determine if catch basins were necessary to meet the City requirements of maintaining one twelve (12) foot driving lane free of flowing or standing water in each traffic direction during the 10-year storm. The analysis did not show a need for the placement of catch basins based on this City criteria. However, catch basins were placed along Eubank Blvd. at the upstream side of an intersection, in order to capture a portion of runoff before it crossed the intersecting street.

10' is adequate

System 1 (Sta. 161+75.00 to 172+45.86)

The proposed drainage structures for this system are a series of Type A catch basins to be located on each side of Eubank Blvd (a total of five catch basins). The catch basins will discharge into an existing 90-inch storm drain. The proposed location for the connection of the catch basins and the 90-inch storm drain is an existing manhole. Because the manhole lies outside the southern limits of the Eubank Boulevard Improvements project, coordination of the connection with the G Avenue reconstruction currently in design by Sandia National Laboratories will be required. This project should make connection

System 2 (Sta. 172+45.86 to 191+59.96)

The east side of System 2 will consist of three Type A catch basins connected to an existing storm drain. Two of the catch basins will be placed along Eubank Blvd., and the third is to be placed at the low point in westbound Innovation Parkway.

The runoff from the south side of Innovation Parkway will flow into a catch basin that will be built with the development of the Sandia Science and Technology Park. This catch basin was not included in these proposed drainage improvements, because it will discharge to a new storm drain system that will not be built with the Eubank Blvd. Improvements.

Two Type A catch basins placed in series along the west side of Eubank Blvd. will discharge into a new 24-inch storm drain. This new storm drain will convey the flow across the roadway and into the existing 84-inch storm drain.

A Double Wing Type A catch basin is also proposed for the low area on the north side of Opportunity Avenue. A catch basin currently exists on the south side of Opportunity Avenue and discharges into an adjacent 84-inch storm drain. The proposed catch basin will connect to an existing 24-inch stub out.

System 3 (Sta. 191+59.96 to 206+29.33)

System 3 is an entirely stand-alone system that will eventually be part of the Gibson Corridor storm drain. As proposed, the east side of the system will consist of a series of Type A catch basins and a section of storm drain that will accept runoff from Eubank Blvd. and the APS property. A Type A catch basin will be placed south of the intersection of Eubank Blvd. and La Entrada and will be connected via an 18-inch storm drain to a Modified Double Wing Type A catch basin located at the low point of the roadway (Sta. 201+38.67). Runoff from the APS property will also enter the modified catch basin and all the runoff from the east side of the roadway and the APS property will discharge into a new 48-inch storm drain.

The 48-inch storm drain will also receive flow from a Double Wing Type A catch basin on the west side of Eubank Blvd. As designed, the new storm drain will be able to convey the additional flow from the APS property when it is developed. According to the Manzano Mesa Master Drainage Study, the APS site will be allowed to discharge at a maximum rate of 55 cfs under developed conditions. The flow within the new storm drain will discharge into a temporary retention pond. Once the Gibson Corridor storm drain is constructed, the pond will no longer be necessary and will be removed.

The pond will be located west of the Eubank and Southern Boulevard intersection. This location was chosen because it will not have an impact on any future development that may occur within the Department of Energy (DOE) property. The proposed pond site is also within the right-of-way that will be dedicated by DOE to the City of Albuquerque for the construction of the future Gibson Corridor Extension. Without the temporary retention pond, basins 5, 6 and the APS property would have no discharge point. Capacity constraints, elevation differences or, significant utility conflicts render connection to either system 2 or 4 unfeasible.

System 4 (Sta. 206+29.33 to 220+52.97)

Proposed improvements for the east side of Eubank Blvd. include removing and replacing the existing Type A catch basins at the intersection of the north Costco Driveway and Eubank Blvd. and at the intersection of the south Home Depot Driveway and Eubank Blvd. The catch basins will be removed and replaced in order to accommodate the Eubank Blvd. widening. Each of the new catch basins will discharge into the existing 54-inch storm drain as they currently do. A Type D catch basin is proposed to capture flow from the low point at Sta. 216+37.00 and convey the runoff north to a new catch basin and ultimately into the existing storm drain.

The west side of Eubank Blvd. will be serviced by Type A catch basins discharging into a new 18-inch lateral storm drain and ultimately into the existing 54-inch storm drain. The 18-inch lateral will be connected to the 54-inch storm drain via an existing 18-inch storm drain. Because the connection is an 18-inch pipe, the proposed lateral storm drain will not be the

typical minimum diameter of 24 inches. This was approved in a phone conversation with Brad Bingham of the City of Albuquerque. The phone conversation record is located in Appendix F. New Type A catch basins will be placed at the intersections of Eubank Blvd. with Trumbull Avenue, Bell Avenue and Acoma Road.

The catch basins at the intersection of Eubank Blvd. and Trumbull Avenue will capture all the flow draining to it and prevent runoff from flowing west along Trumbull Avenue. A catch basin and valley gutter is proposed for the intersection of Eubank Blvd. and Bell Avenue. The valley gutter will convey any residual flow north to the next catch basin along Eubank Blvd at Acoma Road. The existing catch basin at this intersection must be removed and replaced with a Double Wing Type A catch basin to accommodate the sump condition in the area and the Eubank Blvd. widening. Valley gutter and a 6-inch water block are also proposed at Acoma Road to ensure flow is not conveyed west along Acoma Road.

Temporary Retention Pond

The required capacity of the temporary retention pond was determined by computing the volume of runoff entering the pond. The areas draining to the pond are street basins 5, 6 and the offsite property owned by the Albuquerque Public Schools (APS). Basins 5 and 6 were analyzed for fully developed conditions and the APS property was analyzed for existing conditions. The calculations were performed using the Excess Precipitation and Volumetric Runoff (A.5) method in the DPM. The runoff volume was based on a 100-year, 10-day storm as required by City of Albuquerque design criteria for retention pond sizing. The runoff volumes from basins 5, 6 and the APS property are summarized in Table 1.

Table 1 summarizes the land treatment types utilized and the peak flows computed for each of the contributing drainage basins. All of the drainage basins were analyzed assuming fully developed conditions. The APS property was also analyzed for existing conditions, for the purpose of sizing the temporary retention pond. Also included in Table 1 are the runoff volumes for basin 5, 6 and the APS property.

C. Hydraulic Analysis of Proposed Systems

The profiles of each proposed drainage system are located in Appendix D and the hydraulic grade line is displayed on each.

Systems 1-3

A hydraulic grade line analysis of each systems' proposed drainage structures indicates that surcharging will not occur at manhole and inlet locations under the given pressure flow and head-loss conditions.

System 4

Based on the hydraulic grade line analysis of this system, surcharging will occur at multiple inlets and manholes. The two reasons for the surcharge occurring are: lack of capacity within the existing mainline storm drain and additional runoff proposed to be conveyed within the system.

Review of the as-builts for the existing 54-inch storm drain within Eubank Blvd. indicates that the existing storm drain does not have the capacity for the 100-year storm event. A copy of the as-builts for the existing 54-inch storm drain is included in Appendix E. The approved construction plans for the existing mainline storm drain show surcharge occurring at one manhole along the system and the hydraulic grade line at or above ground elevation at numerous locations. This lack of capacity will be further compounded by the proposed additional runoff to be conveyed.

This additional runoff arises from the City's plans to keep storm runoff from flowing west down Trumbull Avenue, Bell Avenue, Acoma Road and Cochiti Road as it currently does during storm events. Ensuring flow is not conveyed down these streets during typical storm events was the priority. Without increasing the size of the existing mainline storm drain, removing the possibility of surcharge occurring during the 100-year storm event is not feasible.

Temporary Retention Pond

The temporary retention pond is a portion of system 3 that will retain all the runoff from the system-3 storm drain. Based on runoff values from basins 5, 6, and the APS property, the required capacity of the pond is 129,500 cubic feet (3.0 acre-feet). Because there are no current plans for development of the APS property, the runoff volume from the site was based

on existing land conditions. It was also assumed that the Gibson Corridor storm drain would have been constructed before any development on the APS property has begun or ponding on the APS site would be required with the development of the site, to allow flows to be discharged at the historic rate. The available ponding depth will be 7 feet, not including the 1-foot of freeboard. The footprint area of the proposed pond will be approximately 0.8 acres.

V. CONCLUSION

The hydraulic analysis of the Eubank Blvd. drainage improvements and the ultimate storm drain design, indicates that systems 1-3 have sufficient capacity for conveying runoff from the 100-year, 6-hour design storm. System 4 does not have the capacity to convey the 100-year storm event but will satisfy the priority of ensuring flow is not conveyed down the streets intersecting Eubank Blvd. on the west side of the roadway.

The floodplain adjacent to Eubank Blvd., north of La Entrada will also be removed by the improvements proposed within this drainage report.

APPENDIX A

EXCERPTS FROM THE "CITY OF ALBUQUERQUE DEVELOPMENT PROCESS MANUAL"

SECTION 22.2, HYDROLOGY

(Chapter 22, Section 2)

of the

DEVELOPMENT PROCESS MANUAL

VOLUME 2, DESIGN CRITERIA
1997 REVISION

CITY OF ALBUQUERQUE, NEW MEXICO

Chapter 22, Section 2, Hydrology was prepared in cooperation with

BERNALILLO COUNTY, NEW MEXICO

and the

ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY

with the text prepared by the

D.P.M. DRAINAGE DESIGN CRITERIA COMMITTEE

The text of the 1997 revision was reformatted from the January 1993 revision. The equations, tabular values, examples and procedures in Chapter 22, Section 2, Hydrology were not changed by the 1997 revision.

PART A - PROCEDURE FOR 40 ACRE AND SMALLER BASINS

A simplified procedure for projects with sub-basins smaller than 40 acres has been developed based on initial abstraction/uniform infiltration precipitation losses and Rational Method procedures. For this procedure, Bernalillo County has been divided into four (4) Precipitation Zones.

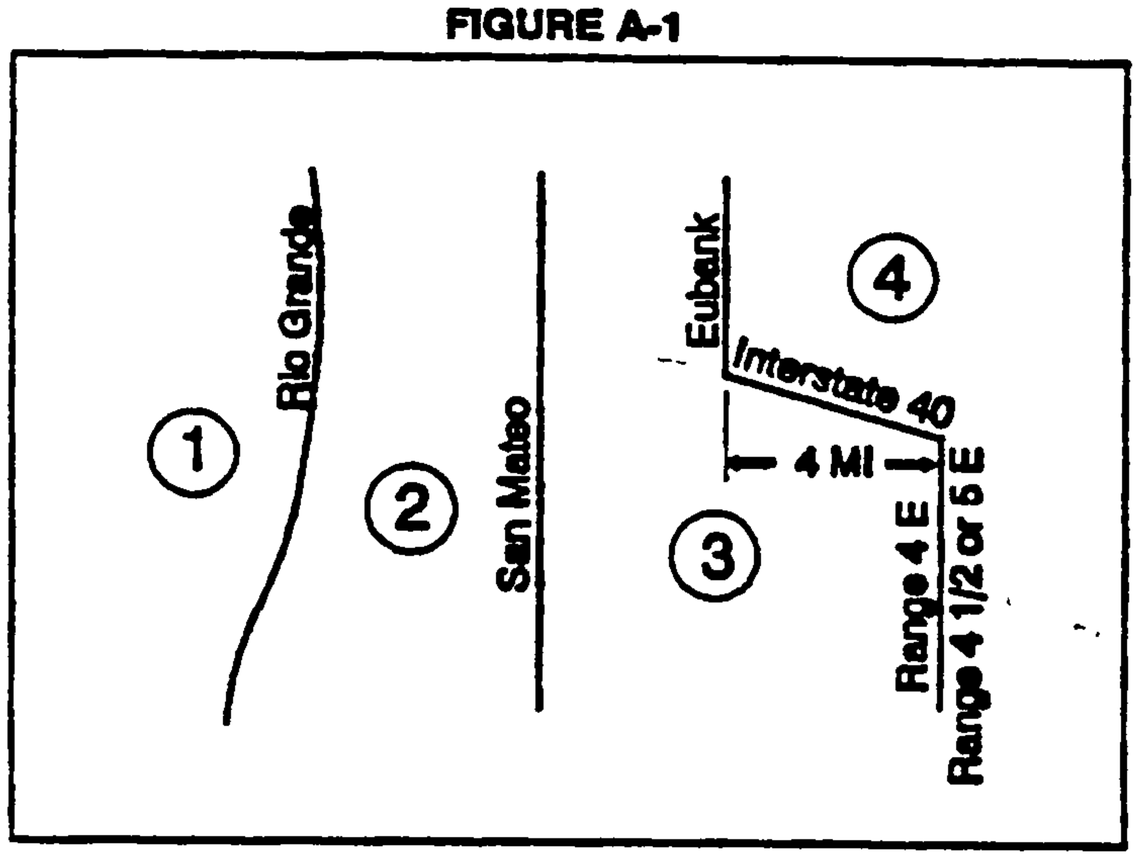
A.1 PRECIPITATION ZONES

Bernalillo County's four precipitation zones are indicated in TABLE A-1 and on FIGURE A-1.

TABLE A-1. PRECIPITATION ZONES

Zone	Location
1	West of the Rio Grande
2	Between the Rio Grande and San Mateo
3	Between San Mateo and Eubank, North of Interstate 40; and between San Mateo and the East boundary of Range 4 East, South of Interstate 40
4	East of Eubank, North of Interstate 40; and East of the East boundary of Range 4 East, South of Interstate 40

FIGURE A-1



Where a watershed extends across a zone boundary, use the zone which contains the largest portion of the watershed.

EXAMPLE A-4

Find the 100-year 24-hour and 4-day runoff volume, V_{1440} and V_{4days}, for the area in Example A-3.

V_{360} = 2.41 acre-feet

V_{1440} = 2.41 + 7 ac * (2.66 - 2.20) / 12 = 2.68 acre-feet

V_{4DAYS} = 2.41 + 7 ac * (3.12 - 2.20) / 12 = 2.95 acre-feet

A.6 PEAK DISCHARGE RATE FOR SMALL WATERSHEDS

Small watersheds are less than or equal to 40 acres.

Peak Discharge

Using a 0.2-hour (12-minute) time of concentration, peak discharge, Q_P, per acre is the volume of excess precipitation in the heaviest 12-minute portion of the storm, divided by the time increment 12 minutes, and multiplied by an attenuation factor. The attenuation factor (0.59 for treatment A, 0.67 for treatment B, 0.75 for treatment C and 0.93 for treatment D) describes the effect of routing. Determine the peak discharge using the values in TABLE A-9, which have been adjusted to consider the effects of initial abstraction.

TABLE A-9. PEAK DISCHARGE (CFS/ACRE)

Zone	Treatment		100-YR [2-YR, 10-YR]	
	A	B	C	D
1	1.29 [0.00, 0.24]	2.03 [0.03, 0.76]	2.87 [0.47, 1.49]	4.37 [1.69, 2.89]
2	1.56 [0.00, 0.38]	2.28 [0.08, 0.95]	3.14 [0.60, 1.71]	4.70 [1.86, 3.14]
3	1.87 [0.00, 0.58]	2.60 [0.21,1.19]	3.45 [0.78, 2.00]	5.02 [2.04, 3.39]
4	2.20 [0.05, 0.87]	2.92 [0.38, 1.45]	3.73 [1.00, 2.26]	5.25 [2.17, 3.57]

TABLE A-4. LAND TREATMENTS	
Treatment	**Land Condition**
A	Soil uncompacted by human activity with 0 to 10 percent slopes. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, groundcover and infiltration capacity.
B	Irrigated lawns, parks and golf courses with 0 to 10 percent slopes. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes greater than 10 percent and less than 20 percent.
C	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Gravel or rock on plastic (desert landscaping). Irrigated lawns and parks with slopes greater than 10 percent. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes at 20 percent or greater. Native grass, weed and shrub areas with clay or clay loam soils and other soils of very low permeability as classified by SCS Hydrologic Soil Group D.
D	Impervious areas, pavement and roofs.
Most watersheds contain a mix of land treatments. To determine proportional treatments, measure respective subareas. In lieu of specific measurement for treatment D, the areal percentages in TABLE A-5 may be employed.	

To determine the peak rate of discharge,

1) Determine the area in each treatment, A_A, A_B, A_C and A_D.

2) Multiply the peak rate for each treatment by the respective areas and sum to compute the total Q_P.

$$\text{Total } Q_P = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D \qquad \text{(a-10)}$$

Example A-5
Find 100-year Q_P for 14 acres in zone 1. The four land treatments are: 3 acres in treatment A, 5 acres in treatment B, 2 acres in treatment C and 4 acres in treatment D. Total Q_P = (1.29 * 3) + (2.03 * 5) + (2.87 * 2) + (4.37 * 4) = 37.24 cfs

3) Approximately the same results can be achieved by a Rational Method solution. The 0.2-hour (12-minute) peak intensities, I, are given in TABLE A-10 and Rational Method coefficients, C, are given in TABLE A-11.

$$\text{Total } Q_P = (C_A * I * A_A) + (C_B * I * A_B) + (C_C * I * A_C) + (C_D * I * A_D) \qquad \text{(a-11)}$$

TABLE A-10. PEAK INTENSITY (IN/HR at t_c = 0.2 hour)

Zone	Intensity 100-YR [2-YR, 10-YR]
1	4.70 [1.84, 3.14]
2	5.05 [2.04, 3.41]
3	5.38 [2.21, 3.65]
4	5.61 [2.34, 3.83]

TABLE A-6. INITIAL ABSTRACTION (IA)	
Treatment	**Initial Abstraction (inches)**
A	0.65
B	0.50
C	0.35
D	0.10

Infiltration is the only significant abstraction after the initial abstraction. After initial abstraction is satisfied, treat infiltration as a constant loss rate as specified in TABLE A-7.

TABLE A-7. INFILTRATION (INF)	
Treatment	**Loss Rate (inches/hour)**
A	1.67
B	1.25
C	0.83
D	0.04*
* Treatment D infiltration rate is applicable from 0 to 3 hours; use uniform reduction from 3 to 6 hours, with no infiltration after 6 hours.	

Runoff from a previous event can saturate a channel bed, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

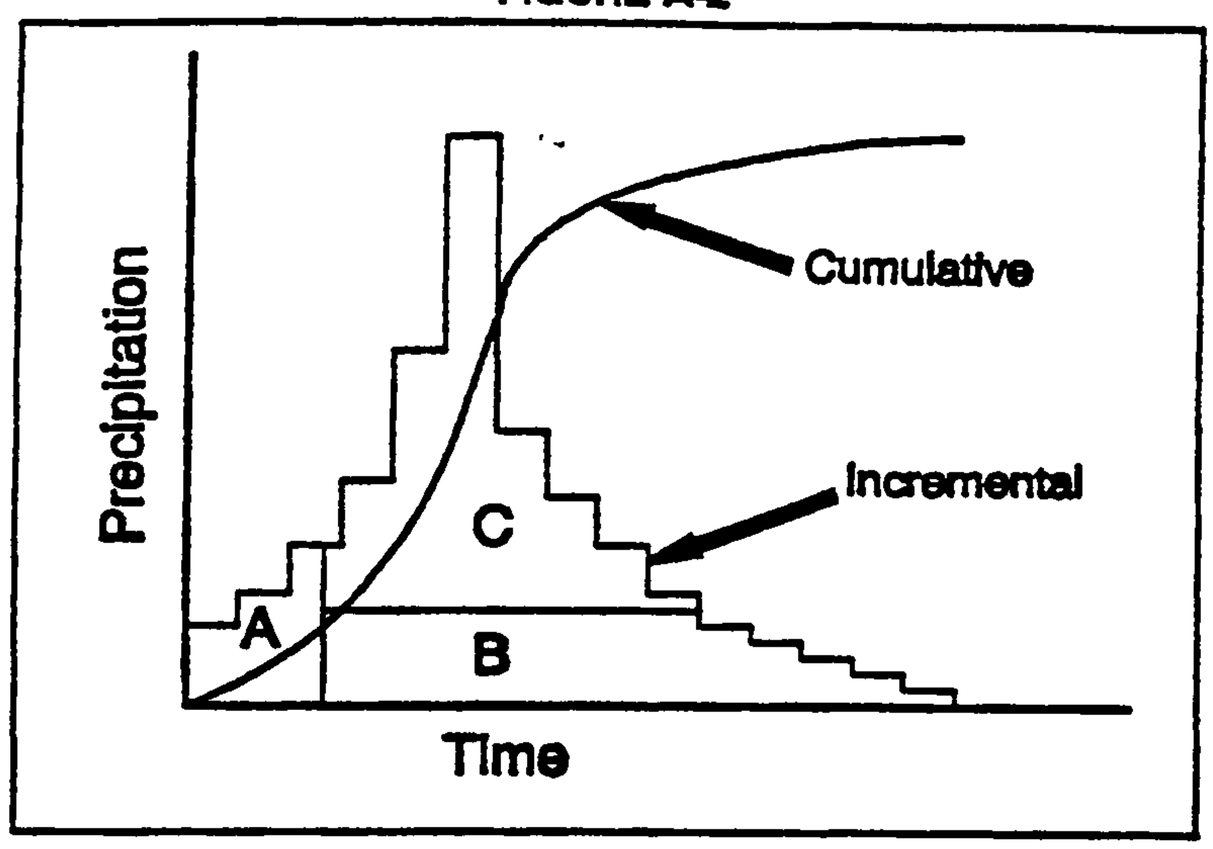
USED FOR POND SIZING

A.5 EXCESS PRECIPITATION & VOLUMETRIC RUNOFF

Excess precipitation, E, is the depth of precipitation remaining after abstractions are removed. Excess precipitation does not depend on watershed area. Excess precipitation is determined by subtracting the initial abstraction and infiltration from the design storm hydrograph. FIGURE A-2 illustrates the development of excess precipitation. The curved line plots cumulative precipitation. Precipitation intensities (in/hr) are shown as a histogram. Initial abstraction is area A. The horizontal line is at a height corresponding to the infiltration rate. Infiltration loss is area B. The remaining histogram, area C, is excess precipitation.

"POND SIZING"

FIGURE A-2



Excess precipitation, E, by zone and treatment is summarized in TABLE A-8.

(NOTE: In this table and several tables which follow, corresponding values for 2- and 10- year storms are shown in brackets below each 100-year value)

TABLE A-8: EXCESS PRECIPITATION, E (INCHES) - 6 HOUR STORM

Zone	Treatment			100-YR [2-YR, 10-YR]
	A	B	C	D
1	0.44 [0.00, 0.08]	0.67 [0.01, 0.22]	0.99 [0.12, 0.44]	1.97 [0.72, 1.24]
2	0.53 [0.00, 0.13]	0.78 [0.02, 0.28]	1.13 [0.15, 0.52]	2.12 [0.79, 1.34]
3	0.66 [0.00, 0.19]	0.92 [0.06, 0.36]	1.29 [0.20, 0.62]	2.36 [0.89, 1.50]

"Pond Sizing"

To determine the volume of runoff,

1) Determine the area in each treatment, A_A, A_B, A_C, A_D

2) Compute the weighted excess precipitation, E

$$\text{Weighted E} = \frac{E_A A_A + E_B A_B + E_C A_C + E_D A_D}{A_A + A_B + A_C + A_D} \qquad \text{(a-5)}$$

3) Multiply the weighted E by the watershed area.

$$V_{360} \text{ (as volume)} = \text{weighted E}^* (A_A + A_B + A_C + A_D) \qquad \text{(a-6)}$$

EXAMPLE A-3
Find the 100-year V_{360} for 30 acres in zone 1. Eight acres are treatment A, 10 acres are treatment B, 5 acres are treatment C, and 7 acres are treatment D.
Weighted E = ((8 * 0.44) + (10 * 0.67) + (5 * 0.99) + (7 * 1.97)) / 30 = 0.965 inches
Volume = (0.965 * 30) / 12 = 2.41 acre-ft. = V_{360}

For ponds which hold water for longer than 6 hours, longer duration storms are required to establish runoff volumes. Since the additional precipitation is assumed to occur over a long period, the additional volume is based on the runoff from the impervious areas only.

For 24-hour storms:

$$V_{1440} = V_{360} + A_D * (P_{1440} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-7)}$$

For 4-day storms:

$$V_{4DAYS} = V_{360} + A_D * (P_{4DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-8)}$$

STORM FOR RETENTION POND SIZING

For 10-day storms:

$$V_{10DAYS} = V_{360} + A_D * (P_{10DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-9)}$$

"Pond Sizing"

"Type B" basins are generally placed downstream of and/or in conjunction with "Type A" basins on streets other than arterials and collectors. This type basin has potential to collect substantial runoff when the grating is clean. If "Type B" basins are used alone, without a "Type A" within 150 feet upstream, the capacity shown in Plate 22.3 D-7 should be reduced 15% due to clogging. "Type B" basins are used with standard 8" curb and gutter.

"Type C" basins are generally placed downstream of and/or in conjunction with "Type A" basins. If "Type C" basins are used without a "Type A" within 150 feet upstream, the capacity shown in Plates 22.3 D-5 and 22.3 D-6 should be reduced 15% for clogging. "Type C" basins are used with standard 8" curb and gutter.

"Type D" basins are generally used on streets with slope greater than 5%, in driveways and in certain valley areas as described above. "Type D" basins can be used with either standard 8" curb and gutter or with mountable curb.

The number of catch basins to be connected in series should not exceed two. If the connection of more than two catch basins in series is unavoidable, consideration should be given to designing a lateral drain.

c. Connector Pipe
   (1) The minimum diameter of connector pipe is 18 inches.
   (2) The horizontal alignment of connector pipes must not contain angle points or bends, unless approved by the City Engineer.
   (3) Connector pipes outletting into a pipe from both sides of a street should be offset 8 feet or more at the main line. Exceptions to this criterion must be approved by the City Engineer. Connections at manholes or junction structures are preferred.
   (4) ~~The minimum length of connector pipe to be installed between catch basins in series shall be 12 feet, unless otherwise approved by the City Engineer.~~ The catch basin spacing shall be a minimum of 25 feet distance between curb transitions.
   (5) Catch basin connector pipes shall outlet at the downstream end of the catch basins, unless prevented by field conditions. Downstream, in this paragraph, refers to the directions of the gutter slope at the catch basin in question.
   (6) Where feasible, connector pipes should be located so as to avoid, as much as possible, cutting into existing cross gutters and spandrels.
   (7) The conversions of "type A's, B's or C's to D's basins will not be permitted. If the catch basin is in conflict with a driveway, the catch basin will be removed and replaced with another inlet outside of the driveway. To avoid conflicts with driveways, the engineer should identify the proposed driveways on the grading plan when catch basins front the lots.

E. <u>Street Hydraulics</u>
   1. A secondary use of the street network is the conveyance of storm runoff. This secondary use must always be subsidiary to the primary function of streets which is the safe conveyance of people and vehicles. The goals of street hydraulic design are therefore:
      a. To provide an economical means of transporting storm runoff.

      b. To ensure that the safety and convenience of the public are preserved.
      c. To prevent storm runoff, once collected by the street system, from leaving the street right-of-way except at specially designated locations.

   2. Street hydraulic design critical are as follows:

a. Manning's roughness coefficient is 0.017.
b. Flow depths plus velocity head ($v^2/2g$) in the event of the 100-year design discharge may not exceed 0.2 feet above curb height ~~or 0.87 feet at any location~~ and shall be contained ~~within the street right-of-way~~.
c. Flow depths in the event of the 10-year design discharge may not exceed 0.5 feet in any collector or arterial street. One ~~twelve (12) foot driving~~ lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.
d. The product of depth times velocity shall not exceed 6.5 in any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet.)

3. For streets with more than two driving lanes in each direction:

   a. The product of depth times velocity may not exceed 6.5 at any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet).
   b. Inverted crown streets are prohibited.
   c. The assumption of equal flow distribution between gutters on undivided streets and between street sections on divided streets is only valid where its validity can be demonstrated.

4. Plates 22.3 D-1 through 22.3 D-4 may be used where applicable in the hydraulic design of streets. T-intersections, radical slope changes and intersections are potential locations for hydraulic jumps when upstream slopes are steeper than critical slope.

   When conditions indicate that a hydraulic jump or that the effects of superelevation will allow runoff to exceed street hydraulic design criteria, provisions must be made to for treatment of the problem. The warping of street sections and the construction of deflector walls for these purposes is prohibited unless specifically authorized by the City Engineer.

5. Intersections and other radical changes in street cross section and slope require special consideration whenever the flow depth/street slope relationship results in flows occurring in the supercritical flow regime. The critical slope line shown on the street rating curves is used to determine on which side of critical depth the flow occurs and if slope or cross section changes will allow the flow to cross through critical depth from supercritical.

   If flow is likely to cross into the subcritical flow range, then Plate 22.3 E-1, "Tail Water vs. Froude Number" is used to determine the height and Plate 22.3 E-2, "Length of Jump vs. Froude Number" figure is used to determine jump length. The height of jump should not exceed 0.2 feet ~~above curb height~~ ~~0.87 feet~~ and ~~the jump should~~ shall be contained within the street right-of-way.

APPENDIX B

PLATES

PLATE 1 – DRAINAGE BASIN MAP
PLATE 2 – DRAINAGE BASIN MAP

APPENDIX C

CALCULATIONS

System 1 – Sta: 161+75.00 to 172+45.86

System 2 – Sta: 172+45.86 to 191+59.96

System 3 – Sta: 191+59.96 to 206+29.33

System 4 – Sta: 206+29.33 to 220+52.97

SYSTEM 1 HYDROLOGY CALCS:

- BASIN #2 (WEST SIDE):

A = 1.82 ACRES

100% "D"

Q = 1.82 * 5.02 = 9.1 cfs

- BASIN #1 (EAST SIDE):

A = 3.64 ACRES

90% "D"

10% "C"

Q = (0.9 * 3.64 * 5.02) + (0.1 * 3.64 * 3.45) = 17.7 cfs

SYSTEM 2

"SYSTEM 2" DRAINAGE BASIN CALCULATIONS:

- **BASIN 3**

AREA = 3.15 ACRES

LAND TREATMENT TYPES: 100% "D"

$Q_{100\text{-}YR}$ = 3.15 * 5.02 = 15.8 cfs

- **BASIN 4**

AREA = 3.21 ACRES

LAND TREATMENT TYPES: 100% "D"

$Q_{100\text{-}YR}$ = 3.21 * 5.02 = 16.1 cfs

"System 3" Drainage Basin Calculations

- **Basin 5**

Area = 2.84 acres
Land treatment types: 100% "D"

$Q_{100\text{-}yr} = 2.84 * 5.02 = \underline{14.3 \text{ cfs}}$

- Basin 5 Runoff Volume:

Excess precipitation = $E = \frac{2.84 * 2.36}{2.84} = 2.36$ inches

$V_{360} = \frac{2.36}{12} * (2.84 * 43{,}560) = 24{,}330 \text{ ft}^3 = 0.56$ acre-feet

$V_{10\text{-days}} = 0.56 + [2.84 * (4.90 - 2.60)/12] = 1.10$ acre-feet $= 48{,}040 \text{ ft}^3$

- **Basin 6**

Area = 2.66 acres
Land treatment types: 100% "D"

$Q_{100\text{-}yr} = 2.66 * 5.02 = \underline{13.4 \text{ cfs}}$

- Basin 5 Runoff Volume:

Excess precipitation = $E = 2.36$ inches

$V_{360} = \frac{2.36}{12} * (2.66 * 43{,}560) = 22{,}790 \text{ ft}^3 = 0.52$ acre-feet

$V_{10\text{-days}} = 0.52 + [2.66 * (4.90 - 2.60)/12] = 1.03$ acre-feet $= 44{,}870 \text{ ft}^3$

SYSTEM 3

· APS PROPERTY (UNDEVELOPED)

AREA = 13.3 ACRES

LAND TREATMENT TYPES : 85% "A"
15% "C"

$Q_{100\text{-}YR} = (0.85 * 13.3 * 1.87) + (0.15 * 13.3 * 3.45) = \underline{28.0 \text{ cfs}}$

· APS PROPERTY (DEVELOPED)

LAND TREATMENT TYPES : 25% "B"
25% "C"
50% "D"

$Q_{100\text{-}YR} = (0.25 * 13.3 * 2.6) + (0.25 * 13.3 * 3.45) * (0.5 * 13.3 * 5.02) = 53.5 \text{ CFS}$

USE $\underline{55 \text{ CFS}}$

· APS PROPERTY RUNOFF VOLUME (UNDEVELOPED)

EXCESS PRECIPITATION = E

$$\text{WEIGHTED } E = \frac{(13.3 * 0.85 * 0.66) + (13.3 * 0.15 * 1.29)}{13.3} = 0.755 \text{ INCHES}$$

$$V_{360} = \frac{0.755}{12} * (13.3 * 43{,}560) = 36{,}450 \text{ FT}^3 = 0.84 \text{ ACRE-FEET}$$

$V_{360} = V_{10\text{-}DAYS}$ BECAUSE THE UNDEVELOPED APS PROPERTY HAS NO LAND TREATMENT OF TYPE "D".

Bohannan ▲ Huston

PROJECT NAME _____ SHEET _____ OF _____

PROJECT NO. _____ BY _____ DATE _____

SUBJECT System 3 CH'D _____ DATE _____

"SYSTEM 3" POND SIZING

ASSUMPTIONS: · APS PROPERTY UNDEVELOPED (13.3 ACRES)
· EUBANK ROADWAY FULLY DEVELOPED (5.5 ACRES)

TREATMENT TYPES:

· APS: 85% "A"
15% "C"

· EUBANK ROADWAY: 100% "D"

EXCESS PRECIPITATION = E

$$\text{WEIGHTED } E = \frac{(0.66 * 13.3 * 0.85) + (1.29 * 13.3 * 0.15) + (2.36 * 5.5)}{18.8}$$

= 1.22 INCHES OF EXCESS PRECIPITATION

$$V_{360} = \frac{1.22}{12} * (18.8 \times 43{,}560) = 83{,}260 \text{ ft}^3 = 1.91 \text{ ACRE FEET}$$

$$V_{10\text{-DAYS}} = 1.91 + [5.5 * (4.90 - 2.60)/12] = 2.97 \text{ ACRE-FEET} = \underline{\underline{129{,}375 \text{ FT}^3}}$$

TEMPORARY RETENTION POND CAPACITY USE 129,500 FT³

Bohannan Huston

PROJECT NAME _____ SHEET _____ OF _____
PROJECT NO _____ BY _____ DATE _____
SUBJECT System 3 _____ CH'D _____ DATE _____

SYSTEM 4

- NORTH OF SOUTHERN BLVD.
- BASIN 7: AREA = 0.614 ACRES

  $Q = (0.614 * 5.02) \simeq$ 3.1 cfs ✓

- BASIN 8: A = 0.429 ACRES

  $Q = (0.429 * 5.02) \simeq$ 2.2 cfs ✓

- BASIN 9: AREA = 1.378 ACRES

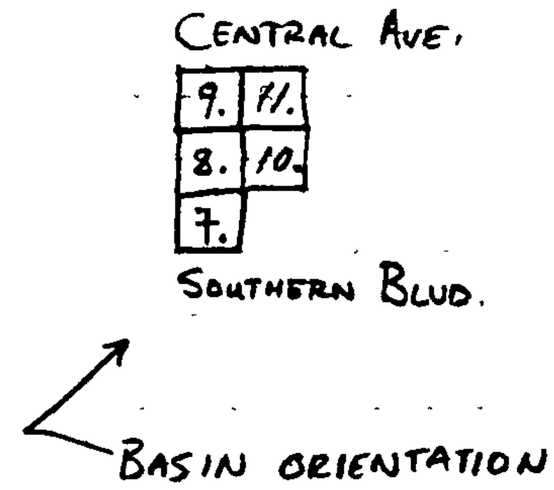
  $Q = (1.378 * 5.02) \simeq$ 6.9 cfs ✓

- BASIN 10: AREA 2.300 ACRES

  $Q = (0.287 * 5.02) \simeq$ 11.5 cfs ✓

- BASIN 11: AREA = 1.184 ACRES

  $Q = (1.184 * 5.02) \simeq$ 6.0 cfs ✓



- 100-YR DESIGN STORM
- ZONE 3

- BASIN 10 AND 11 CONTRIBUTE FLOWS ENTERING CATCH BASINS AT SOUTHERN "HOME DEPOT" ENTRANCE.

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .005

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.17	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.28	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.36	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.44	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.51	0.78	0.00	0.05
0.06	0.06	0.03	0.0	0.99	0.58	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.64	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.70	1.25	0.01	0.09
0.09	0.09	0.06	0.0	1.48	0.76	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.81	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	0.86	1.72	0.01	0.12
0.12	0.12	0.11	0.1	1.97	0.92	1.88	0.01	0.13
0.13	0.13	0.13	0.1	2.14	0.97	2.03	0.01	0.14
0.14	0.14	0.16	0.1	2.66	0.93	2.54	0.01	0.15
0.15	0.15	0.18	0.2	3.18	0.92	3.06	0.01	0.16
0.16	0.16	0.22	0.2	3.70	0.93	3.57	0.01	0.17
0.17	0.17	0.25	0.2	4.22	0.95	4.08	0.01	0.18
0.18	0.18	0.30	0.3	4.73	0.98	4.59	0.01	0.19
0.19	0.19	0.35	0.3	5.25	1.01	5.10	0.02	0.21
0.20	0.20	0.40	0.4	5.77	1.04	5.61	0.02	0.22
0.21	0.21	0.46	0.5	6.29	1.08	6.12	0.02	0.23
0.22	0.22	0.52	0.6	6.81	1.12	6.64	0.02	0.24
0.23	0.23	0.59	0.7	7.33	1.15	7.15	0.02	0.25
0.24	0.24	0.67	0.8	7.85	1.19	7.66	0.02	0.26
0.25	0.25	0.74	0.9	8.37	1.23	8.17	0.02	0.27
0.26	0.26	0.83	1.1	8.89	1.27	8.68	0.03	0.29
0.27	0.27	0.92	1.2	9.41	1.31	9.19	0.03	0.30
0.28	0.28	1.01	1.4	9.93	1.35	9.70	0.03	0.31
0.29	0.29	1.11	1.5	10.45	1.39	10.21	0.03	0.32
0.30	0.30	1.22	1.7	10.97	1.43	10.73	0.03	0.33
0.31	0.31	1.33	1.9	11.48	1.47	11.24	0.03	0.34
0.32	0.32	1.44	2.2	12.00	1.50	11.75	0.04	0.36
0.33	0.33	1.56	2.4	12.52	1.54	12.26	0.04	0.37
0.34	0.34	1.69	2.7	13.04	1.58	12.77	0.04	0.38
0.35	0.35	1.82	2.9	13.56	1.62	13.28	0.04	0.39
0.36	0.36	1.95	3.2	14.08	1.66	13.79	0.04	0.40
0.37	0.37	2.09	3.5	14.60	1.69	14.31	0.04	0.41
0.38	0.38	2.24	3.9	15.12	1.73	14.82	0.05	0.43
0.39	0.39	2.39	4.2	15.64	1.77	15.33	0.05	0.44
0.40	0.40	2.54	4.6	16.16	1.80	15.84	0.05	0.45
0.41	0.41	2.71	5.0	16.68	1.84	16.35	0.05	0.46
0.42	0.42	2.87	5.4	17.20	1.87	16.86	0.05	0.47
0.43	0.43	3.04	5.8	17.72	1.91	17.37	0.06	0.49
0.44	0.44	3.22	6.3	18.23	1.95	17.89	0.06	0.50
0.45	0.45	3.40	6.7	18.75	1.98	18.40	0.06	0.51

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	7.7	19.79	2.05	19.42	0.07	0.54
0.48	0.48	3.98	8.3	20.29	2.09	19.91	0.07	0.55
0.49	0.49	4.18	8.9	20.79	2.12	20.40	0.07	0.56
0.50	0.50	4.38	9.4	21.29	2.16	20.89	0.07	0.57
0.51	0.51	4.59	10.1	21.78	2.19	21.38	0.07	0.58

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0066

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.20	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.32	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.42	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.51	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.59	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.66	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.73	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.80	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	0.87	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.93	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	0.99	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.05	1.88	0.02	0.14
0.13	0.13	0.13	0.1	2.14	1.11	2.03	0.02	0.15
0.14	0.14	0.16	0.2	2.66	1.07	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.06	3.06	0.02	0.17
0.16	0.16	0.22	0.2	3.70	1.07	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.09	4.08	0.02	0.19
0.18	0.18	0.30	0.3	4.73	1.12	4.59	0.02	0.20
0.19	0.19	0.35	0.4	5.25	1.16	5.10	0.02	0.21
0.20	0.20	0.40	0.5	5.77	1.20	5.61	0.02	0.22
0.21	0.21	0.46	0.6	6.29	1.24	6.12	0.02	0.23
0.22	0.22	0.52	0.7	6.81	1.28	6.64	0.03	0.25
0.23	0.23	0.59	0.8	7.33	1.33	7.15	0.03	0.26
0.24	0.24	0.67	0.9	7.85	1.37	7.66	0.03	0.27
0.25	0.25	0.74	1.1	8.37	1.41	8.17	0.03	0.28
0.26	0.26	0.83	1.2	8.89	1.46	8.68	0.03	0.29
0.27	0.27	0.92	1.4	9.41	1.51	9.19	0.04	0.31
0.28	0.28	1.01	1.6	9.93	1.55	9.70	0.04	0.32
0.29	0.29	1.11	1.8	10.45	1.60	10.21	0.04	0.33
0.30	0.30	1.22	2.0	10.97	1.64	10.73	0.04	0.34
0.31	0.31	1.33	2.2	11.48	1.68	11.24	0.04	0.35
0.32	0.32	1.44	2.5	12.00	1.73	11.75	0.05	0.37
0.33	0.33	1.56	2.8	12.52	1.77	12.26	0.05	0.38
0.34	0.34	1.69	3.1	13.04	1.82	12.77	0.05	0.39
0.35	0.35	1.82	3.4	13.56	1.86	13.28	0.05	0.40
0.36	0.36	1.95	3.7	14.08	1.90	13.79	0.06	0.42
0.37	0.37	2.09	4.1	14.60	1.95	14.31	0.06	0.43
0.38	0.38	2.24	4.4	15.12	1.99	14.82	0.06	0.44
0.39	0.39	2.39	4.8	15.64	2.03	15.33	0.06	0.45
0.40	0.40	2.54	5.3	16.16	2.07	15.84	0.07	0.47
0.41	0.41	2.71	5.7	16.68	2.11	16.35	0.07	0.48
0.42	0.42	2.87	6.2	17.20	2.15	16.86	0.07	0.49
0.43	0.43	3.04	6.7	17.72	2.19	17.37	0.07	0.50
0.44	0.44	3.22	7.2	18.23	2.23	17.89	0.08	0.52
0.45	0.45	3.40	7.7	18.75	2.28	18.40	0.08	0.53

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	8.9	19.79	2.35	19.42	0.09	0.56
0.48	0.48	3.98	9.5	20.29	2.40	19.91	0.09	0.57
0.49	0.49	4.18	10.2	20.79	2.44	20.40	0.09	0.58
0.50	0.50	4.38	10.9	21.29	2.48	20.89	0.10	0.60
0.51	0.51	4.59	11.6	21.78	2.52	21.38	0.10	0.61

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .007

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.21	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.33	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.43	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.52	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.60	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.68	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.76	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.83	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	0.89	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.96	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	1.02	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.08	1.88	0.02	0.14
0.13	0.13	0.13	0.2	2.14	1.14	2.03	0.02	0.15
0.14	0.14	0.16	0.2	2.66	1.10	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.09	3.06	0.02	0.17
0.16	0.16	0.22	0.2	3.70	1.10	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.13	4.08	0.02	0.19
0.18	0.18	0.30	0.3	4.73	1.16	4.59	0.02	0.20
0.19	0.19	0.35	0.4	5.25	1.19	5.10	0.02	0.21
0.20	0.20	0.40	0.5	5.77	1.23	5.61	0.02	0.22
0.21	0.21	0.46	0.6	6.29	1.28	6.12	0.03	0.24
0.22	0.22	0.52	0.7	6.81	1.32	6.64	0.03	0.25
0.23	0.23	0.59	0.8	7.33	1.37	7.15	0.03	0.26
0.24	0.24	0.67	0.9	7.85	1.41	7.66	0.03	0.27
0.25	0.25	0.74	1.1	8.37	1.46	8.17	0.03	0.28
0.26	0.26	0.83	1.2	8.89	1.50	8.68	0.04	0.30
0.27	0.27	0.92	1.4	9.41	1.55	9.19	0.04	0.31
0.28	0.28	1.01	1.6	9.93	1.60	9.70	0.04	0.32
0.29	0.29	1.11	1.8	10.45	1.64	10.21	0.04	0.33
0.30	0.30	1.22	2.1	10.97	1.69	10.73	0.04	0.34
0.31	0.31	1.33	2.3	11.48	1.73	11.24	0.05	0.36
0.32	0.32	1.44	2.6	12.00	1.78	11.75	0.05	0.37
0.33	0.33	1.56	2.9	12.52	1.83	12.26	0.05	0.38
0.34	0.34	1.69	3.2	13.04	1.87	12.77	0.05	0.39
0.35	0.35	1.82	3.5	13.56	1.91	13.28	0.06	0.41
0.36	0.36	1.95	3.8	14.08	1.96	13.79	0.06	0.42
0.37	0.37	2.09	4.2	14.60	2.00	14.31	0.06	0.43
0.38	0.38	2.24	4.6	15.12	2.05	14.82	0.07	0.45
0.39	0.39	2.39	5.0	15.64	2.09	15.33	0.07	0.46
0.40	0.40	2.54	5.4	16.16	2.13	15.84	0.07	0.47
0.41	0.41	2.71	5.9	16.68	2.18	16.35	0.07	0.48
0.42	0.42	2.87	6.4	17.20	2.22	16.86	0.08	0.50
0.43	0.43	3.04	6.9	17.72	2.26	17.37	0.08	0.51
0.44	0.44	3.22	7.4	18.23	2.30	17.89	0.08	0.52
0.45	0.45	3.40	8.0	18.75	2.34	18.40	0.09	0.54

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	9.2	19.79	2.42	19.42	0.09	0.56
0.48	0.48	3.98	9.8	20.29	2.47	19.91	0.09	0.57
0.49	0.49	4.18	10.5	20.79	2.51	20.40	0.10	0.59
0.50	0.50	4.38	11.2	21.29	2.55	20.89	0.10	0.60
0.51	0.51	4.59	11.9	21.78	2.59	21.38	0.10	0.61

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0091

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.24	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.37	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.49	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.59	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.69	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.78	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.86	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.94	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	1.02	1.41	0.02	0.11
0.10	0.10	0.08	0.1	1.64	1.09	1.56	0.02	0.12
0.11	0.11	0.09	0.1	1.81	1.17	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.24	1.88	0.02	0.14
0.13	0.13	0.13	0.2	2.14	1.30	2.03	0.03	0.16
0.14	0.14	0.16	0.2	2.66	1.25	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.24	3.06	0.02	0.17
0.16	0.16	0.22	0.3	3.70	1.26	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.28	4.08	0.03	0.20
0.18	0.18	0.30	0.4	4.73	1.32	4.59	0.03	0.21
0.19	0.19	0.35	0.5	5.25	1.36	5.10	0.03	0.22
0.20	0.20	0.40	0.6	5.77	1.41	5.61	0.03	0.23
0.21	0.21	0.46	0.7	6.29	1.45	6.12	0.03	0.24
0.22	0.22	0.52	0.8	6.81	1.50	6.64	0.04	0.26
0.23	0.23	0.59	0.9	7.33	1.56	7.15	0.04	0.27
0.24	0.24	0.67	1.1	7.85	1.61	7.66	0.04	0.28
0.25	0.25	0.74	1.2	8.37	1.66	8.17	0.04	0.29
0.26	0.26	0.83	1.4	8.89	1.71	8.68	0.05	0.31
0.27	0.27	0.92	1.6	9.41	1.77	9.19	0.05	0.32
0.28	0.28	1.01	1.8	9.93	1.82	9.70	0.05	0.33
0.29	0.29	1.11	2.1	10.45	1.87	10.21	0.05	0.34
0.30	0.30	1.22	2.3	10.97	1.93	10.73	0.06	0.36
0.31	0.31	1.33	2.6	11.48	1.98	11.24	0.06	0.37
0.32	0.32	1.44	2.9	12.00	2.03	11.75	0.06	0.38
0.33	0.33	1.56	3.2	12.52	2.08	12.26	0.07	0.40
0.34	0.34	1.69	3.6	13.04	2.13	12.77	0.07	0.41
0.35	0.35	1.82	4.0	13.56	2.18	13.28	0.07	0.42
0.36	0.36	1.95	4.4	14.08	2.23	13.79	0.08	0.44
0.37	0.37	2.09	4.8	14.60	2.28	14.31	0.08	0.45
0.38	0.38	2.24	5.2	15.12	2.33	14.82	0.08	0.46
0.39	0.39	2.39	5.7	15.64	2.38	15.33	0.09	0.48
0.40	0.40	2.54	6.2	16.16	2.43	15.84	0.09	0.49
0.41	0.41	2.71	6.7	16.68	2.48	16.35	0.10	0.51
0.42	0.42	2.87	7.3	17.20	2.53	16.86	0.10	0.52
0.43	0.43	3.04	7.8	17.72	2.58	17.37	0.10	0.53
0.44	0.44	3.22	8.4	18.23	2.62	17.89	0.11	0.55
0.45	0.45	3.40	9.1	18.75	2.67	18.40	0.11	0.56

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	10.4	19.79	2.76	19.42	0.12	0.59
0.48	0.48	3.98	11.2	20.29	2.81	19.91	0.12	0.60
0.49	0.49	4.18	11.9	20.79	2.86	20.40	0.13	0.62
0.50	0.50	4.38	12.7	21.29	2.91	20.89	0.13	0.63
0.51	0.51	4.59	13.6	21.78	2.95	21.38	0.14	0.65

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0121

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.27	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.43	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.57	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.68	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.79	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.90	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.99	1.09	0.02	0.09
0.08	0.08	0.05	0.1	1.32	1.09	1.25	0.02	0.10
0.09	0.09	0.06	0.1	1.48	1.18	1.41	0.02	0.11
0.10	0.10	0.08	0.1	1.64	1.26	1.56	0.02	0.12
0.11	0.11	0.09	0.1	1.81	1.34	1.72	0.03	0.14
0.12	0.12	0.11	0.2	1.97	1.42	1.88	0.03	0.15
0.13	0.13	0.13	0.2	2.14	1.50	2.03	0.04	0.17
0.14	0.14	0.16	0.2	2.66	1.45	2.54	0.03	0.17
0.15	0.15	0.18	0.3	3.18	1.43	3.06	0.03	0.18
0.16	0.16	0.22	0.3	3.70	1.45	3.57	0.03	0.19
0.17	0.17	0.25	0.4	4.22	1.48	4.08	0.03	0.20
0.18	0.18	0.30	0.5	4.73	1.52	4.59	0.04	0.22
0.19	0.19	0.35	0.5	5.25	1.57	5.10	0.04	0.23
0.20	0.20	0.40	0.6	5.77	1.62	5.61	0.04	0.24
0.21	0.21	0.46	0.8	6.29	1.68	6.12	0.04	0.25
0.22	0.22	0.52	0.9	6.81	1.74	6.64	0.05	0.27
0.23	0.23	0.59	1.1	7.33	1.79	7.15	0.05	0.28
0.24	0.24	0.67	1.2	7.85	1.85	7.66	0.05	0.29
0.25	0.25	0.74	1.4	8.37	1.92	8.17	0.06	0.31
0.26	0.26	0.83	1.6	8.89	1.98	8.68	0.06	0.32
0.27	0.27	0.92	1.9	9.41	2.04	9.19	0.06	0.33
0.28	0.28	1.01	2.1	9.93	2.10	9.70	0.07	0.35
0.29	0.29	1.11	2.4	10.45	2.16	10.21	0.07	0.36
0.30	0.30	1.22	2.7	10.97	2.22	10.73	0.08	0.38
0.31	0.31	1.33	3.0	11.48	2.28	11.24	0.08	0.39
0.32	0.32	1.44	3.4	12.00	2.34	11.75	0.09	0.41
0.33	0.33	1.56	3.7	12.52	2.40	12.26	0.09	0.42
0.34	0.34	1.69	4.1	13.04	2.46	12.77	0.09	0.43
0.35	0.35	1.82	4.6	13.56	2.52	13.28	0.10	0.45
0.36	0.36	1.95	5.0	14.08	2.58	13.79	0.10	0.46
0.37	0.37	2.09	5.5	14.60	2.63	14.31	0.11	0.48
0.38	0.38	2.24	6.0	15.12	2.69	14.82	0.11	0.49
0.39	0.39	2.39	6.6	15.64	2.75	15.33	0.12	0.51
0.40	0.40	2.54	7.1	16.16	2.80	15.84	0.12	0.52
0.41	0.41	2.71	7.7	16.68	2.86	16.35	0.13	0.54
0.42	0.42	2.87	8.4	17.20	2.92	16.86	0.13	0.55
0.43	0.43	3.04	9.0	17.72	2.97	17.37	0.14	0.57
0.44	0.44	3.22	9.7	18.23	3.03	17.89	0.14	0.58
0.45	0.45	3.40	10.5	18.75	3.08	18.40	0.15	0.60

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	12.0	19.79	3.19	19.42	0.16	0.63
0.48	0.48	3.98	12.9	20.29	3.24	19.91	0.16	0.64
0.49	0.49	4.18	13.8	20.79	3.30	20.40	0.17	0.66
0.50	0.50	4.38	14.7	21.29	3.35	20.89	0.17	0.67
0.51	0.51	4.59	15.7	21.78	3.41	21.38	0.18	0.69

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0157

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.31	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.49	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.64	0.47	0.01	0.04
0.04	0.04	0.01	0.0	0.66	0.78	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.91	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	1.02	0.94	0.02	0.08
0.07	0.07	0.04	0.0	1.15	1.13	1.09	0.02	0.09
0.08	0.08	0.05	0.1	1.32	1.24	1.25	0.02	0.10
0.09	0.09	0.06	0.1	1.48	1.34	1.41	0.03	0.12
0.10	0.10	0.08	0.1	1.64	1.44	1.56	0.03	0.13
0.11	0.11	0.09	0.1	1.81	1.53	1.72	0.04	0.15
0.12	0.12	0.11	0.2	1.97	1.62	1.88	0.04	0.16
0.13	0.13	0.13	0.2	2.14	1.71	2.03	0.05	0.18
0.14	0.14	0.16	0.3	2.66	1.65	2.54	0.04	0.18
0.15	0.15	0.18	0.3	3.18	1.63	3.06	0.04	0.19
0.16	0.16	0.22	0.4	3.70	1.65	3.57	0.04	0.20
0.17	0.17	0.25	0.4	4.22	1.69	4.08	0.04	0.21
0.18	0.18	0.30	0.5	4.73	1.73	4.59	0.05	0.23
0.19	0.19	0.35	0.6	5.25	1.79	5.10	0.05	0.24
0.20	0.20	0.40	0.7	5.77	1.85	5.61	0.05	0.25
0.21	0.21	0.46	0.9	6.29	1.91	6.12	0.06	0.27
0.22	0.22	0.52	1.0	6.81	1.98	6.64	0.06	0.28
0.23	0.23	0.59	1.2	7.33	2.04	7.15	0.06	0.29
0.24	0.24	0.67	1.4	7.85	2.11	7.66	0.07	0.31
0.25	0.25	0.74	1.6	8.37	2.18	8.17	0.07	0.32
0.26	0.26	0.83	1.9	8.89	2.25	8.68	0.08	0.34
0.27	0.27	0.92	2.1	9.41	2.32	9.19	0.08	0.35
0.28	0.28	1.01	2.4	9.93	2.39	9.70	0.09	0.37
0.29	0.29	1.11	2.7	10.45	2.46	10.21	0.09	0.38
0.30	0.30	1.22	3.1	10.97	2.53	10.73	0.10	0.40
0.31	0.31	1.33	3.4	11.48	2.60	11.24	0.10	0.41
0.32	0.32	1.44	3.8	12.00	2.67	11.75	0.11	0.43
0.33	0.33	1.56	4.3	12.52	2.73	12.26	0.12	0.45
0.34	0.34	1.69	4.7	13.04	2.80	12.77	0.12	0.46
0.35	0.35	1.82	5.2	13.56	2.87	13.28	0.13	0.48
0.36	0.36	1.95	5.7	14.08	2.93	13.79	0.13	0.49
0.37	0.37	2.09	6.3	14.60	3.00	14.31	0.14	0.51
0.38	0.38	2.24	6.9	15.12	3.07	14.82	0.15	0.53
0.39	0.39	2.39	7.5	15.64	3.13	15.33	0.15	0.54
0.40	0.40	2.54	8.1	16.16	3.19	15.84	0.16	0.56
0.41	0.41	2.71	8.8	16.68	3.26	16.35	0.16	0.57
0.42	0.42	2.87	9.5	17.20	3.32	16.86	0.17	0.59
0.43	0.43	3.04	10.3	17.72	3.38	17.37	0.18	0.61
0.44	0.44	3.22	11.1	18.23	3.45	17.89	0.18	0.62
0.45	0.45	3.40	11.9	18.75	3.51	18.40	0.19	0.64

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	13.7	19.79	3.63	19.42	0.20	0.67
0.48	0.48	3.98	14.7	20.29	3.69	19.91	0.21	0.69
0.49	0.49	4.18	15.7	20.79	3.76	20.40	0.22	0.71
0.50	0.50	4.38	16.7	21.29	3.82	20.89	0.23	0.73
0.51	0.51	4.59	17.8	21.78	3.88	21.38	0.23	0.74

Single A inlet, in sump condition with curb openings on both sides:
Open Area (for orifice calc in sq. ft.): 3.9314236
Length of Weir (feet): 11.354167

Head (ft)	Head (in)	Weir Q	Orifice Q	Control Q
0.05	0.6	0.34	4.23	0.34
0.1	1.2	0.96	5.99	0.96
0.15	1.8	1.77	7.33	1.77
0.2	2.4	2.72	8.47	2.72
0.25	3	3.80	9.46	3.80
0.3	3.6	5.00	10.37	5.00
0.35	4.2	6.30	11.20	6.30
0.4	4.8	7.70	11.97	7.70
0.45	5.4	9.19	12.70	9.19
0.5	6	10.76	13.39	10.76
0.55	6.6	12.41	14.04	12.41
0.6	7.2	14.14	14.66	14.14
0.65	7.8	15.95	15.26	15.26
0.67	8.04	16.69	15.49	15.49
0.7	8.4	17.82	15.84	15.84
0.75	9	19.76	16.39	16.39
0.8	9.6	21.77	16.93	16.93
0.85	10.2	23.85	17.45	17.45
0.9	10.8	25.98	17.96	17.96
0.95	11.4	28.18	18.45	18.45
1	12	30.43	18.93	18.93
1.05	12.6	32.74	19.40	19.40
1.1	13.2	35.11	19.85	19.85
1.15	13.8	37.53	20.30	20.30
1.2	14.4	40.00	20.74	20.74

Calculation of open area:

	(in^2)	(ft^2)
Total Grate Area	1000	6.9444444
Cross Bar Area	-366	-2.541667
Supports (ends)	-115.625	-0.802951
Areas Counted Twice	47.75	0.3315972
	566.125	3.9314236

Calculation of Length of Weir:

	(in)	(ft)
Total Perimeter of Grate	90	7.5
Short Cross Bars	-3.5	-0.291667
End Supports	-9.25	-0.770833
Bearing Bars	-13	-1.083333
Curb Openings	72	6
	136.25	11.354167

Single D inlet, in sump condition:

Open Area (for orifice calc in sq. ft.): 3.93

Length of Weir (feet): 8.40

Head (ft)	Head (in)	Weir Q	Orifice Q	Control Q
0.05	0.6	0.25	4.23	0.25
0.1	1.2	0.71	5.99	0.71
0.15	1.8	1.31	7.33	1.31
0.2	2.4	2.01	8.47	2.01
0.25	3	2.81	9.46	2.81
0.3	3.6	3.70	10.37	3.70
0.35	4.2	4.66	11.20	4.66
0.4	4.8	5.69	11.97	5.69
0.45	5.4	6.79	12.70	6.79
0.5	6	7.96	13.39	7.96
0.55	6.6	9.18	14.04	9.18
0.6	7.2	10.46	14.66	10.46
0.65	7.8	11.79	15.26	11.79
0.67	8.04	12.34	15.49	12.34
0.7	8.4	13.18	15.84	13.18
0.75	9	14.61	16.39	14.61
0.8	9.6	16.10	16.93	16.10
0.85	10.2	17.63	17.45	17.45
0.9	10.8	19.21	17.96	17.96
0.95	11.4	20.83	18.45	18.45
1	12	22.50	18.93	18.93
1.05	12.6	24.21	19.40	19.40
1.1	13.2	25.96	19.85	19.85
1.15	13.8	27.75	20.30	20.30
1.2	14.4	29.58	20.74	20.74
1.25	15	31.45	21.16	21.16
1.3	15.6	33.35	21.58	21.58

Calculation of open area:

	(in^2)	(ft^2)
Total Grate Area	1000	6.944
Cross Bar Area	-366	-2.542
Supports (ends)	-115.625	-0.803
Areas Counted Twice	47.75	0.332
	566.125	3.931

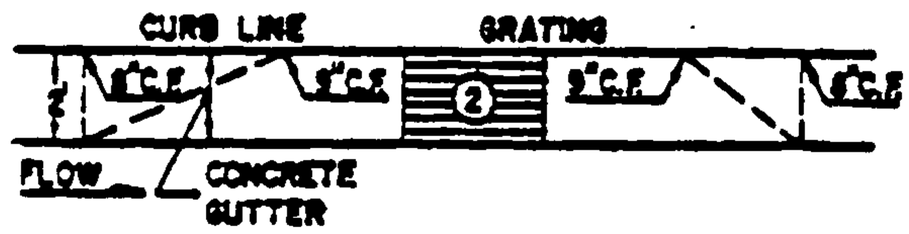
Calculation of Length of Weir (four sides open):

	(in)	(ft)
Total Perimeter of Grate	130	10.833
Short Cross Bars	-7	-0.583
End Supports	-9.25	-0.771
Bearing Bars	-13	-1.083
	100.75	8.396
		0.775

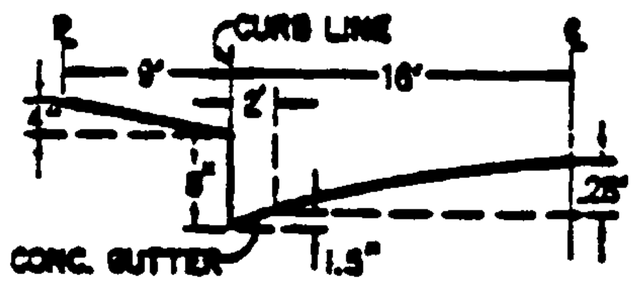
Calculation of Length of Weir (three sides open):

	(in)	(ft)
Total Perimeter of Grate	90	7.500
Short Cross Bars	-3.5	-0.292
End Supports	-9.25	-0.771
Bearing Bars	-13	-1.083
	64.25	5.354
		0.714

GRATING CAPACITIES FOR TYPE 'A', 'C' and 'D'



GRATING & GUTTER PLAN



TYPICAL HALF STREET SECTION
(ABOVE BASIN)

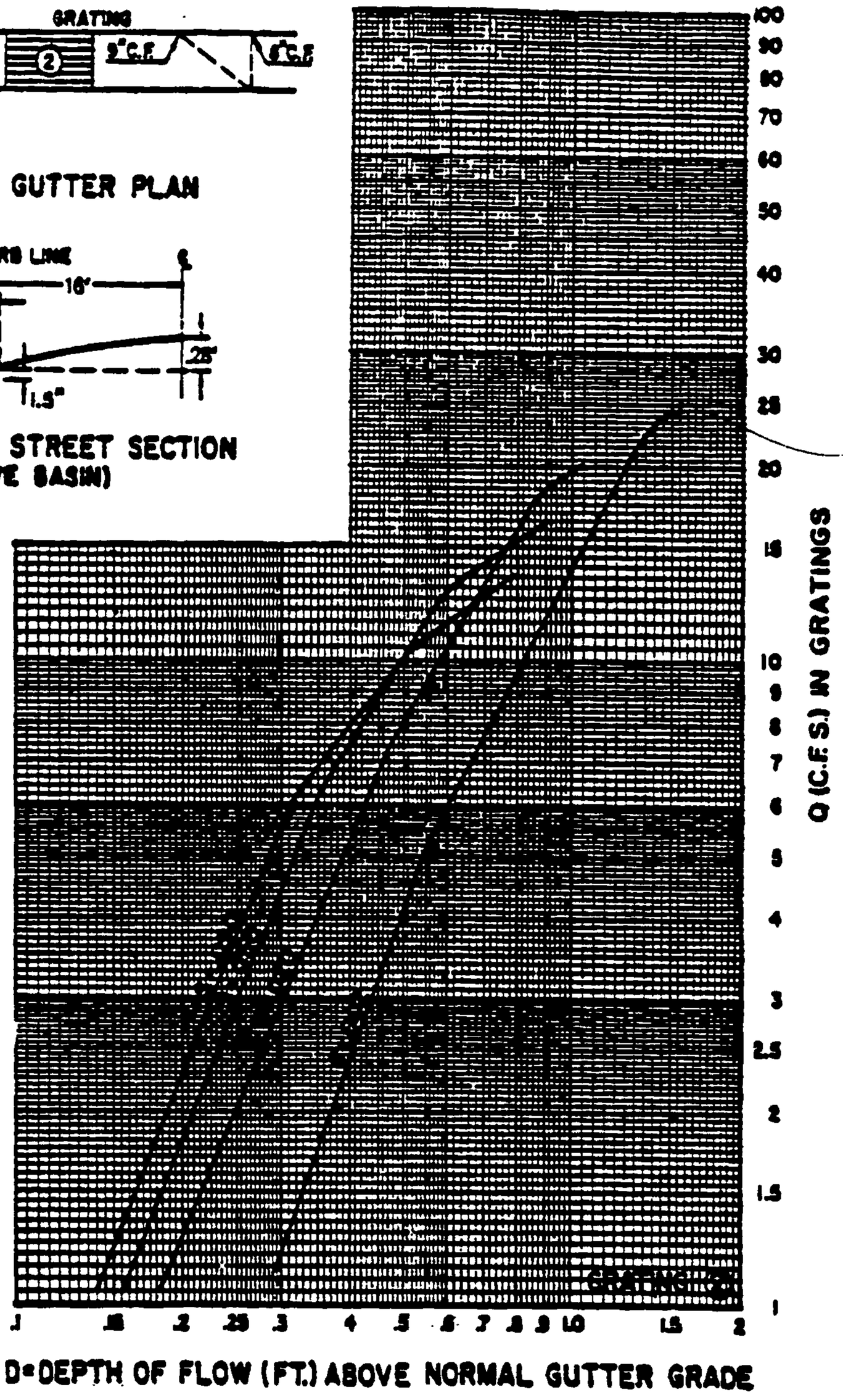
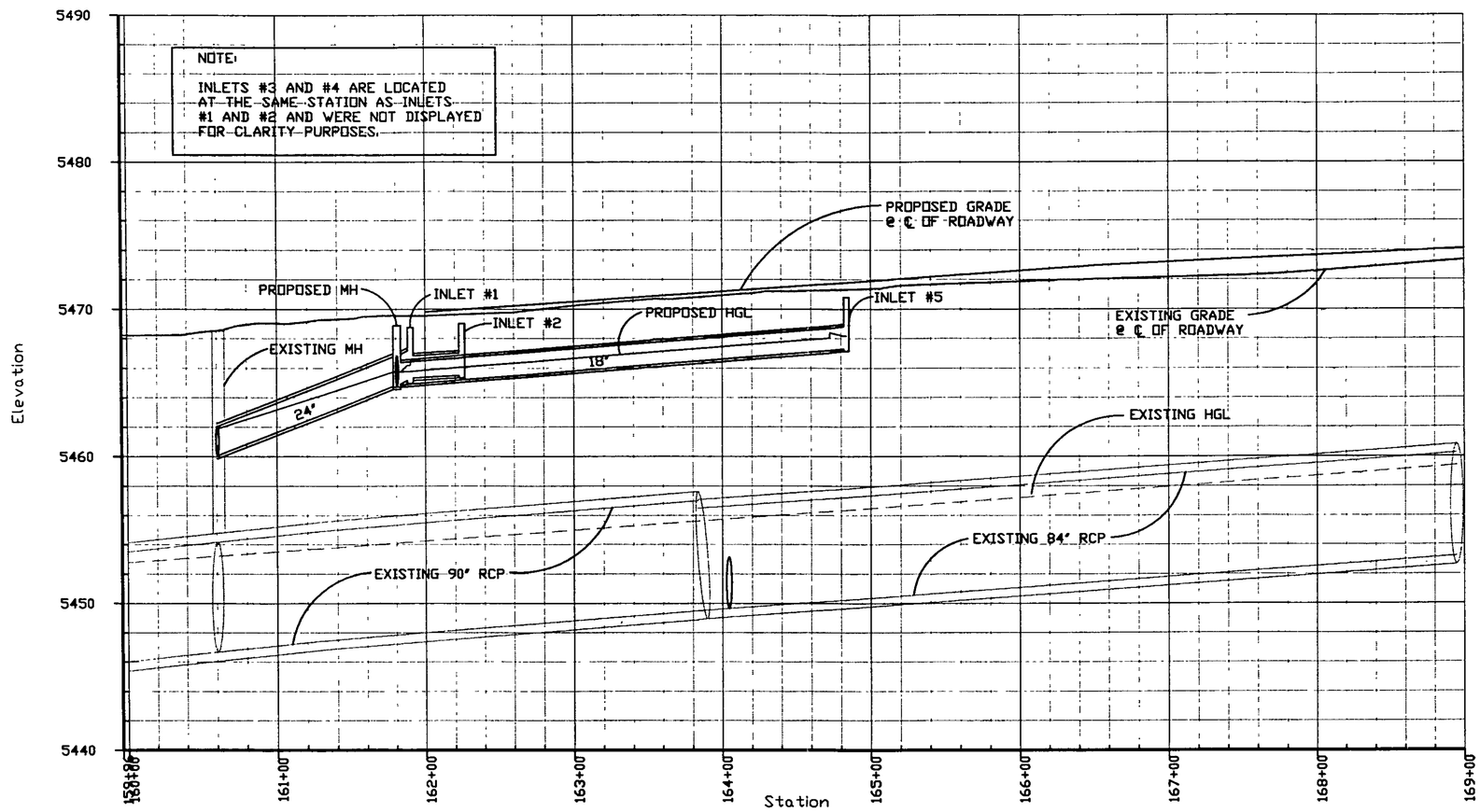


PLATE 22.3 D-5

APPENDIX D

Hydraulic Grade Line Profiles

Bohannan ▲ Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS



SCALE: 1" = 100' (HORIZ)
1" = 10' (VERT)

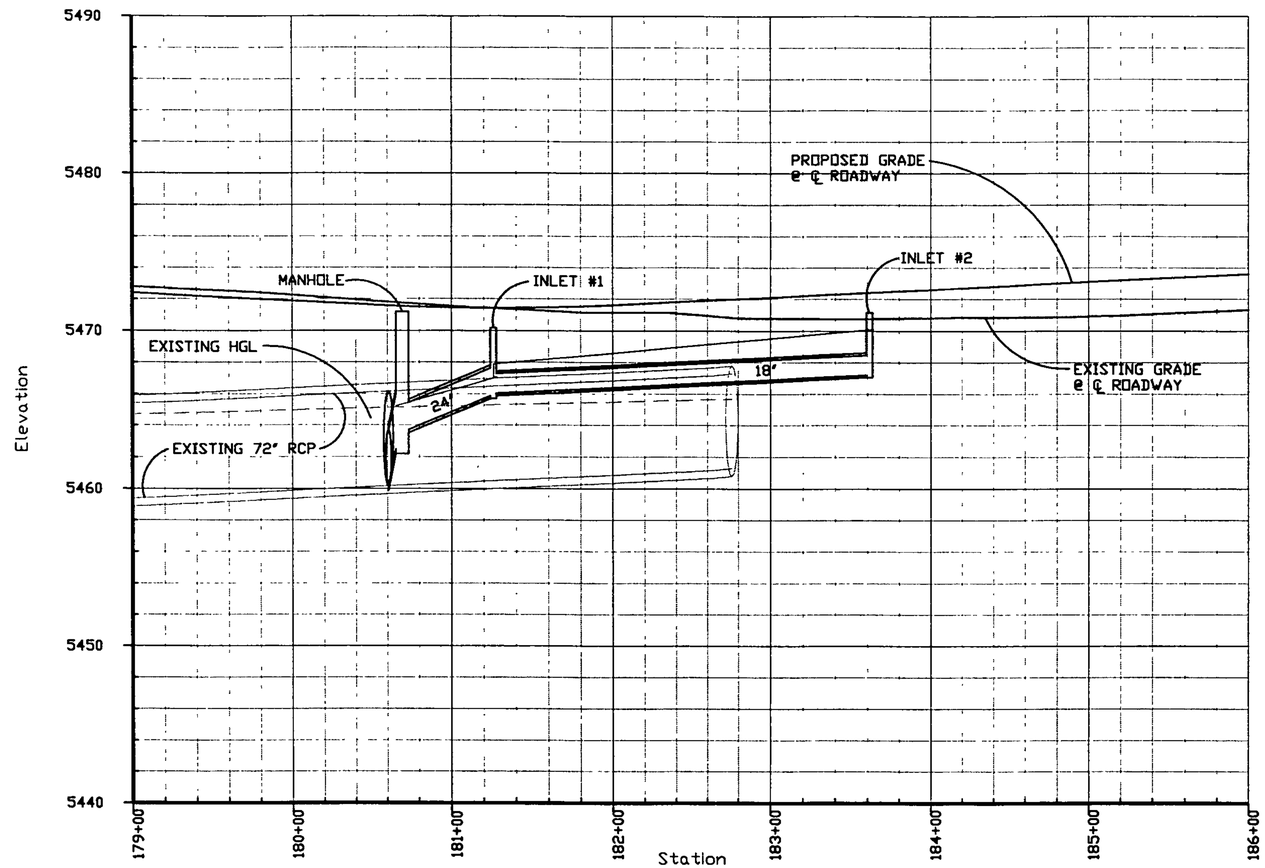
LINE	SURFACE	OFFSET
———	00268 Survey Update	0.00
———	top	0.00

Scaled 10.0000 Times Ver.
Scaled 1.0000 Times Hor.

SYSTEM 1

P:\00268\Hydro\Reports\syst.dwg
January 02, 2002 - 8:27am
kkorkel

Bohannan ▲ Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS



SCALE: 1" = 100' (HORIZ)
1" = 10' (VERT)

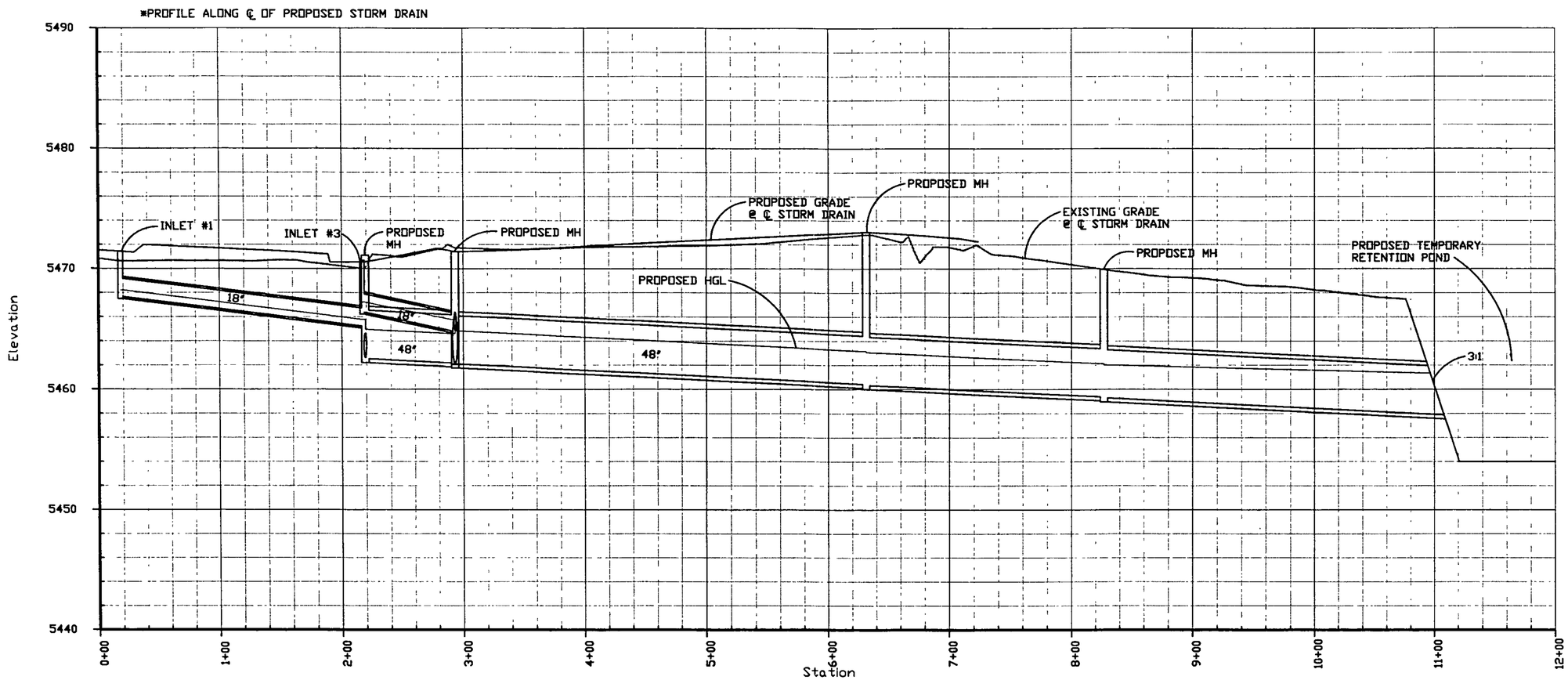
LINE	SURFACE	OFFSET
———	00268 Survey Update	0.00
———	top	0.00

Scaled 10.0000 Times Ver.
Scaled 1.0000 Times Hor.

SYSTEM 2

P:\00268\Hydro\Reports\sys2.dwg
January 02, 2002 - 8:28am
Mortel

Bohannan ▲ Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS



SCALE: 1" = 100' (HORIZ)
1" = 10' (VERT)

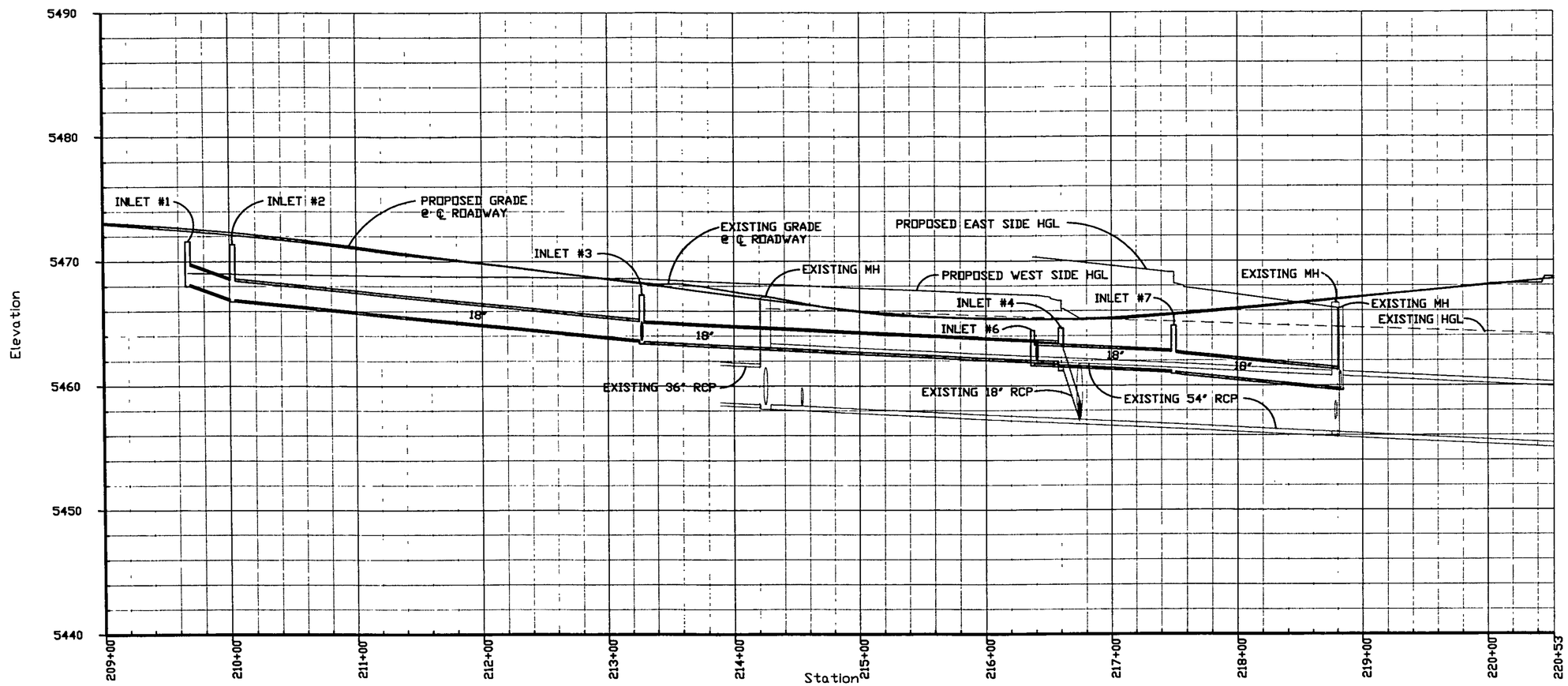
LINE	SURFACE	OFFSET
———	00268 Survey Update	0.00
———	top	0.00

Scaled 10.0000 Times Ver.
Scaled 1.0000 Times Hor.

SYSTEM 3

P:\00268\Hydro\Reports\sys3.dwg
January 02, 2002 - 2:48pm
Model

Bohannan ▲ Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS



SCALE: 1" = 100' (HORIZ)
1" = 10' (VERT)

LINE	SURFACE	OFFSET
———	00268 Survey Update	0.00
———	top	0.00

Scaled 10.0000 Times Ver.
Scaled 1.0000 Times Hor.

SYSTEM 4

P:\00268\Hydro\Reports\sys4.dwg
January 02, 2002 - 8:29am

APPENDIX E

Eubank Blvd. 54-inch Storm Drain As-Builts

APPENDIX F

Correspondence

TELEPHONE CONFERENCE NOTES

JOB NO. 00268

PROJECT TITLE Eubank

DATE **9/24/01**

TIME 11:40

CONVERSATION WITH Brad Bingham **OF** City of Albuquerque

AND John Gurule **OF** BHI

SUBJECT **Size of trunk line in area north of Southern Ave.**

CONTENT

I asked Brad if it was acceptable to use an 18 inch diameter pipe for the trunk line in the area north of Southern Avenue instead of the standard 24 inch pipe. His response was, "since the trunk line would be connecting to an existing 18-inch pipe in the Acoma area, it would not be beneficial to the City, to place a 24-inch pipe upstream and then narrow it down to 18 inches at the connection."

John Gurule

BOHANNAN HUSTON, INC.

Courtyard One

7500 JEFFERSON NE

Albuquerque

NEW MEXICO 87109-4335

voice 505.823.1000

fax 505.798.7988

www.bhinc.com

L21-052

EUBANK BOULEVARD IMPROVEMENTS, KIRTLAND AIR FORCE BASE TO CENTRAL AVENUE DRAINAGE REPORT

Gloria Sedeño

Return TO:
BRAD BINGHAM
1 STOP SHOP
FROM: PAT
STOVALL

July 9, 2002

PREPARED FOR:

CITY OF ALBUQUERQUE
P.O. BOX 1293
ALBUQUERQUE, NM 87103

Bohannan △ Huston INC.

EUBANK BOULEVARD IMPROVEMENTS,
KIRTLAND AIR FORCU BASE TO CENTRAL AVENUE
DRAINAGE REPORT

I, Craig W. Hoover, hereby certify that I am a Register Professional Engineer, registered in the state of New Mexico, and that the following report was prepared under my direction and is true and correct to the best of my knowledge and belief.

Craig W. Hoover 7/9/02

Craig W. Hoover, P.E.

NMPE No. 11848

CRAIG W. HOOVER
NEW MEXICO
11848
REGISTERED PROFESSIONAL ENGINEER

TABLE OF CONTENTS

I. INTRODUCTION 1
II. METHODOLOGIES AND REFERENCES 1
III. EXISTING HYDRAULIC/HYDROLOGIC CONDITIONS 1
IV. STORM DRAIN DESIGN 2
A. Hydrologic Analysis and Results 2
B. Proposed Drainage Improvements 3
C. Hydraulic Analysis of Proposed Systems 8
V. CONCLUSION 10

TABLES

TABLE 1 3
TABLE 2 4

APPENDICES

APPENDIX A - Excerpts from the "City of Albuquerque Development Process Manual"

APPENDIX B - Plates

PLATE 1 – Drainage Basin Map

PLATE 2 – Drainage Basin Map

APPENDIX C - Calculations

Drainage Basin Hydrology Calculations

Typical Street Section

Street Section Capacity Calculations

Sump Inlet (Type A and Type D) Capacity Calculations

On-Grade Inlet Capacity Nomograph

APPENDIX D - Hydraulic Grade Line Profiles

APPENDIX E - Eubank Boulevard. 54-inch Storm Drain As-Builts

APPENDIX F - Correspondence

I. INTRODUCTION

The purpose of this report is to address the drainage improvements for Eubank Boulevard from Kirtland Air Force Base to Central Avenue. Included in this report:

- data supporting the proposed Eubank Boulevard. storm drain improvements.
- data supporting the proposed size and location of the temporary retention pond.

The basis for this report is the approved "Manzano Mesa Master Drainage Study", Smith Engineering Company, August 1996.

II. METHODOLOGIES AND REFERENCES

This report is prepared in Accordance with City of Albuquerque Development Process Manual (DPM) criteria; specifically, the procedures outlined in DPM Section 22.2. This report also references the approved "Manzano Mesa Master Drainage Study," Smith Engineering Company, August 1996.

III. EXISTING HYDRAULIC/HYDROLOGIC CONDITIONS

The existing hydraulic conditions along the Eubank Boulevard Improvement Project vary greatly from the northern section of the project to the southern. The property adjacent to Eubank Boulevard from Southern Boulevard to Central Avenue is currently developed and serviced by an existing storm drain system that conveys flows north toward Central Avenue. Curb and gutter exists along this portion of Eubank Boulevard and conveys runoff to catch basins connected to the storm drain.

Eubank Boulevard from Southern Boulevard south to Kirtland AFB is mainly undeveloped with scattered areas of development existing along the east side of the roadway. There is currently no curb and gutter along this section of roadway except for approximately 1400 feet of curb and gutter located from Opportunity Avenue southward. A storm drain parallels Eubank Boulevard from the future Innovation Parkway (Stephen Moody Street) and continues south, past Research Road. This storm drain, designated as System "A" in the Manzano Mesa Master Drainage Plan, currently

receives flow from developments in the area and will also ultimately drain a portion of Central Avenue east of Elizabeth Street.

A FEMA Floodplain of approximately 2.2 acres exists on two parcels of land adjacent to Eubank Boulevard. The land containing the northern portion of the floodplain is owned by the Albuquerque Public Schools (APS) and the southern section of the floodplain is within land belonging to GSL Properties of Portland Oregon. In the near future, the southern parcel of land will be the site of the Manzano Mesa Apartments. The proposed drainage improvements for the apartment complex will remove the floodplain located on the southern parcel of land. The storm drain improvements proposed in this drainage report will remove a majority of the floodplain within the APS property.

IV. STORM DRAIN DESIGN

A. Hydrologic Analysis and Results

The hydrology for the fully developed conditions model was based on the City of Albuquerque Development Process Manual, Section 22.2, Hydrology (DPM). The 10-year storm was the basis for analyzing the street hydraulics and the proposed storm drain was sized to accommodate the 100-year storm. The procedure described in Part A of the DPM was used for the analysis, and excerpts of this procedure are located in Appendix A. This procedure is an accepted method for calculating runoff volumes and peak flow rates from drainage basins that are 40 acres or smaller.

The procedure is based on the Rational Method and initial abstraction/uniform infiltration precipitation losses and utilizes four different land treatments. The land treatments range from undisturbed land (Type A) to impervious surfaces (Type D). Table 1 summarizes the land treatment types utilized for each contributing drainage basin in the fully developed conditions model. Each of the street drainage basins, with the exception of the APS property and basin 1, contains minimal areas considered to be Land Treatment Type C (median

landscaping), but this area was determined to be negligible, and the street basins were therefore modeled as being 100% Land Treatment Type D.

TABLE 1

LAND TREATMENT TYPES AND BASIN FLOWS - FULLY DEVELOPED CONDITIONS

DRAINAGE BASIN	TOTAL AREA (ACRES)	LAND TREATMENT TYPES (%)				PEAK FLOW (CFS)	RUNOFF VOLUME (CF)*
		A	B	C	D		
1	2.53	0	0	10	90	17.7	
2	1.82	0	0	0	100	9.1	
3	3.15	0	0	0	100	14.8	
4	3.21	0	0	0	100	16.1	
5	2.84	0	0	0	100	14.3	48040
6	2.66	0	0	0	100	13.4	45000
7	0.61	0	0	0	100	3.1	
8	0.43	0	0	0	100	2.2	
9	1.38	0	0	0	100	6.9	
10	2.3	0	0	0	100	11.5	
11	1.18	0	0	0	100	6.0	
APS Property	13.30	0	25	25	50	55.0	
APS Property**	13.30	85	0	15	0	28.0	36450

* 100-yr 10 day volume
** Existing conditions assumed for this basin only.

This method also takes into account the Precipitation Zone that the project lies within. The entire Eubank Boulevard project lies within Precipitation Zone 3 as defined by the DPM.

Plates 1 and 2 in Appendix C contain the fully developed conditions basin maps.

B. Proposed Drainage Improvements

The proposed Eubank Boulevard storm drain improvements that will be constructed in two phases were divided into 4 systems based on the locations of high points along Eubank

Boulevard. Table 2 summarizes the basins drained by each of the systems and corresponding phases. Plates 1 and 2 display the limits of each system and related drainage basins. Plans for Phase 1 have been completed. Phase 2 will begin in the near future when funding permits.

TABLE 2

DRAINAGE SYSTEMS

PHASE	DRAINAGE SYSTEM	BASINS DRAINED BY EACH SYSTEM
1	1	1, 2
1	2	3, 4
1	3	5, 6, APS
2	4	7, 8, 9, 10, 11

A cross section of the street within each of the system limits was analyzed to determine if catch basins were necessary to meet the City requirements of maintaining one twelve (12) foot driving lane free of flowing or standing water in each traffic direction during the 10-year storm. The analysis did not show a need for the placement of catch basins based on this City criteria. However, catch basins were placed along Eubank Boulevard at the upstream side of intersections, in order to capture a portion of runoff before it crossed the intersecting street.

Phase 1 (Sta. 161+90.57 to Sta. 210+25.83)

Phase 1 incorporates improvements to Eubank Boulevard starting approximately at the entrance to Kirtland AFB to the south and ending approximately at the intersection of Southern Boulevard to the north.

System 1 (Sta. 161+90.78 to 164+86.89)

The proposed drainage structures for this system are a series of Type A catch basins to be located on each side of Eubank Boulevard (a total of five catch basins). The catch basins will discharge into an existing 90-inch storm drain. The proposed location for the connection of the catch basins and the 90-inch storm drain is an existing manhole. Because the manhole lies outside the southern limits of the Eubank Boulevard Improvements project, coordination

of the connection with the G Avenue reconstruction currently in design by Sandia National Laboratories will be required.

System 2 (Sta. 173+72.05 to 183+50.00)

The east side of System 2 will consist of three Type A catch basins connected to an existing storm drain. Two of the catch basins will be placed along Eubank Boulevard, and the third is to be placed at the low point in westbound Innovation Parkway.

The runoff from the south side of Innovation Parkway will flow into a catch basin that will be built with the development of the Sandia Science and Technology Park. This catch basin was not included in these proposed drainage improvements, because it will discharge to a new storm drain system that will not be built with the Eubank Boulevard Improvements.

Two Type A catch basins placed in series along the west side of Eubank Boulevard will discharge into a new 24-inch storm drain. This new storm drain will convey the flow across the roadway and into the existing 84-inch storm drain.

A Double Wing Type A catch basin is also proposed for the low area on the north side of Opportunity Avenue. A catch basin currently exists on the south side of Opportunity Avenue and discharges into an adjacent 84-inch storm drain. The proposed catch basin will connect to an existing 24-inch stub out.

System 3 (Sta. 199+38.47 to 206+09.82 on Eubank and Sta. 0+00 to 8+71.97 on Southern)

System 3 is an entirely stand-alone system that will eventually be part of the Gibson Corridor storm drain. As proposed, the east side of the system will consist of a series of Type A catch basins and a section of storm drain that will accept runoff from Eubank Boulevard and the APS property. A Type A catch basin will be placed south of the intersection of Eubank Boulevard and La Entrada Avenue and will be connected via an 18-inch storm drain to a Modified Double Wing Type A catch basin located at the low point of the roadway (Sta.

201+38.54). Runoff from the APS property will be collected within a depressed area and drained utilizing a ported riser connected with a 30" trunk line to the main 48" storm drain line. A modified storm drain manhole with an inlet cover at the start of the 48" line will collect runoff from the east side of Eubank Boulevard.

The 48-inch storm drain will also receive flow from a Double Wing Type A catch basin on the west side of Eubank Boulevard. As designed, the new storm drain will be able to convey the flow for fully developed conditions from the APS property. According to the Manzano Mesa Master Drainage Study, the APS site will be allowed to discharge at a maximum rate of 55 cfs under developed conditions and 28 cfs under existing conditions. The flow within the new storm drain will discharge into a temporary retention pond. Once the Gibson Corridor storm drain is constructed, the pond will no longer be necessary and will be removed.

The pond will be located west of the Eubank and Southern Boulevard intersection. This location was chosen because it will not have an impact on any future development that may occur within the Department of Energy (DOE) property. The proposed pond site is within the right-of-way that has been dedicated by DOE to the City of Albuquerque for the construction of the future Gibson Corridor Extension. Without the temporary retention pond, basins 5, 6 and the APS property would have no discharge point. Capacity constraints, elevation differences or, significant utility conflicts render connection to either system 2 or 4 unfeasible.

The required capacity of the temporary retention pond was determined by computing the volume of runoff entering the pond. The areas draining to the pond are street basins 5, 6 and the offsite property owned by the Albuquerque Public Schools (APS). Basins 5 and 6 were analyzed for fully developed conditions and the APS property was analyzed for existing conditions. Because there are no current plans for development of the APS property. It was also assumed that the Gibson Corridor storm drain would have been constructed before any development on the APS property has begun or ponding on the APS site would be required with the development of the site, to allow flows to be discharged at the historic (existing conditions) rate. The calculations were performed using the Excess Precipitation and

Volumetric Runoff (A.5) method in the DPM. The runoff volume was based on a 100-year, 10-day storm as required by City of Albuquerque design criteria for retention pond sizing. The runoff volumes from basins 5, 6 and the APS property are summarized in Table 1.

Phase 2

Phase 2 begins approximately at Southern Boulevard to the south and ends approximately at Central Avenue.

System 4 (Sta. 210+25.83 to 220+52.97)

Proposed improvements for the east side of Eubank Boulevard. include removing and replacing the existing Type A catch basins at the intersection of the north Costco Driveway and Eubank Boulevard. and at the intersection of the south Home Depot Driveway and Eubank Boulevard. The catch basins will be removed and replaced in order to accommodate the Eubank Boulevard. widening. Each of the new catch basins will discharge into the existing 54-inch storm drain as they currently do. A Type D catch basin is proposed to capture flow from the low point at Sta. 216+37.00 and convey the runoff north to a new catch basin and ultimately into the existing storm drain.

The west side of Eubank Boulevard. will be serviced by Type A catch basins discharging into a new 18-inch lateral storm drain and ultimately into the existing 54-inch storm drain. The 18-inch lateral will be connected to the 54-inch storm drain via an existing 18-inch storm drain. Because the connection is an 18-inch pipe, the proposed lateral storm drain will not be the typical minimum diameter of 24 inches. This was approved in a phone conversation with Brad Bingham of the City of Albuquerque. The phone conversation record is located in Appendix F. New Type A catch basins will be placed at the intersections of Eubank Boulevard. with Trumbull Avenue, Bell Avenue and Acoma Road.

The catch basins at the intersection of Eubank Boulevard. and Trumbull Avenue will capture all the flow draining to it and prevent runoff from flowing west along Trumbull Avenue.

A catch basin and valley gutter is proposed for the intersection of Eubank Boulevard. and Bell Avenue. The valley gutter will convey any residual flow north to the next catch basin along Eubank Boulevard at Acoma Road. The existing catch basin at this intersection must be removed and replaced with a Double Wing Type A catch basin to accommodate the sump condition in the area and the Eubank Boulevard. widening. Valley gutter and a 6-inch water block are also proposed at Acoma Road to ensure flow is not conveyed west along Acoma Road.

C. Hydraulic Analysis of Proposed Systems

The profiles of each proposed drainage system are located in Appendix D and the hydraulic grade line is displayed on each.

Systems 1-3 (Phase 1)

A hydraulic grade line analysis of each systems' proposed drainage structures indicates that surcharging will not occur at manhole and inlet locations under the given pressure flow and head-loss conditions.

Temporary Retention Pond

The temporary retention pond is a portion of system 3 that will retain all the runoff from the system-3 storm drain. Based on runoff values from basins 5, 6, and the APS property, the required capacity of the pond is 129,500 cubic feet (3.0 acre-feet). The available ponding depth will be 8 feet, not including the 1-foot of freeboard. The footprint area of the proposed pond will be approximately 0.8 acres.

Accumulated storm water within the temporary retention pond is required to be pumped since the pond must not store water longer than 192 hours (2 x 96 hours). The State Engineers Office normally requires retention ponds be drained/pumped within 96 hours or else water rights equaling the amount of stored water must be used. However, based on conversations with the State Engineers Office regarding this project they indicated the City

would be granted twice the normal period or 192 hours due to the temporary nature of the facility.

The depth within the pond resulting from a 2-year storm was determined to be 2.7' (32.5 inches). Within 192 hours, approximately 2 inches of water within the pond would evaporate and 24 inches would be lost due to infiltration for a total of 26 inches. Consequently for any storms slightly less than the 2-year storm or larger pumping will be required.

System 4 (Phase 2)

Based on the hydraulic grade line analysis of this system, surcharging will occur at multiple inlets and manholes. The two reasons for the surcharge occurring are: lack of capacity within the existing mainline storm drain and additional runoff proposed to be conveyed within the system.

Review of the as-builts for the existing 54-inch storm drain within Eubank Boulevard indicates that the existing storm drain does not have the capacity for the 100-year storm event. A copy of the as-builts for the existing 54-inch storm drain is included in Appendix E. The approved construction plans for the existing mainline storm drain show surcharge occurring at one manhole along the system and the hydraulic grade line at or above ground elevation at numerous locations. This lack of capacity will be further compounded by the proposed additional runoff to be conveyed.

This additional runoff arises from the City's plans to keep storm runoff from flowing west down Trumbull Avenue, Bell Avenue, Acoma Road and Cochiti Road as it currently does during storm events. Ensuring flow is not conveyed down these streets during typical storm events was the priority. Without increasing the size of the existing mainline storm drain, removing the possibility of surcharge occurring during the 100-year storm event is not feasible.

V. CONCLUSION

The hydraulic analysis of the Eubank Boulevard drainage improvements and the ultimate storm drain design, indicates that systems 1-3 have sufficient capacity for conveying runoff from the 100-year, 6-hour design storm. System 4 does not have the capacity to convey the 100-year storm event but will satisfy the priority of ensuring flow is not conveyed down the streets intersecting Eubank Boulevard on the west side of the roadway.

The floodplain adjacent to Eubank Boulevard, north of La Entrada Avenue will also be removed by the improvements proposed within this drainage report.

APPENDIX A

EXCERPTS FROM THE "CITY OF ALBUQUERQUE DEVELOPMENT PROCESS MANUAL"

SECTION 22.2, HYDROLOGY

(Chapter 22, Section 2)

of the

DEVELOPMENT PROCESS MANUAL

VOLUME 2, DESIGN CRITERIA
1997 REVISION

CITY OF ALBUQUERQUE, NEW MEXICO

Chapter 22, Section 2, Hydrology was prepared in cooperation with

BERNALILLO COUNTY, NEW MEXICO

and the

ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY

with the text prepared by the

D.P.M. DRAINAGE DESIGN CRITERIA COMMITTEE

The text of the 1997 revision was reformatted from the January 1993 revision. The equations, tabular values, examples and procedures in Chapter 22, Section 2, Hydrology were not changed by the 1997 revision.

PART A - PROCEDURE FOR 40 ACRE AND SMALLER BASINS

A simplified procedure for projects with sub-basins smaller than 40 acres has been developed based on initial abstraction/uniform infiltration precipitation losses and Rational Method procedures. For this procedure, Bernalillo County has been divided into four (4) Precipitation Zones.

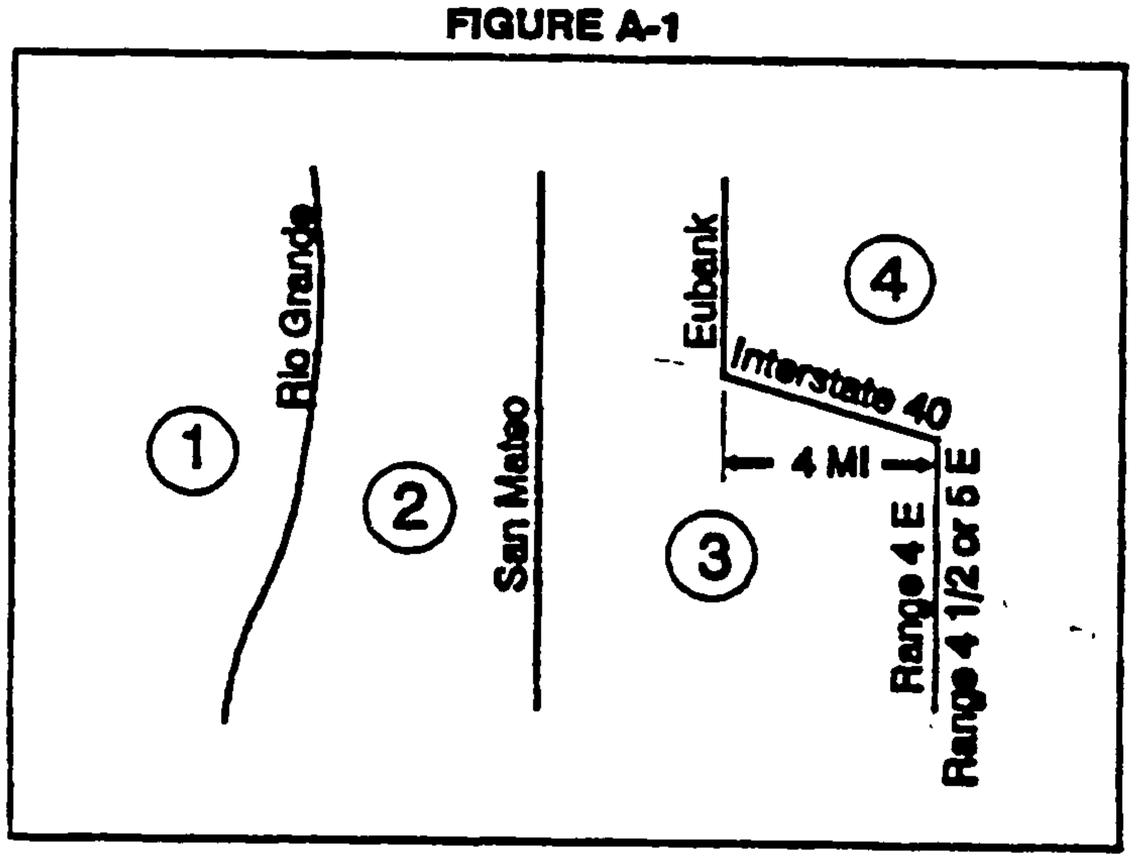
A.1 PRECIPITATION ZONES

Bernalillo County's four precipitation zones are indicated in TABLE A-1 and on FIGURE A-1.

TABLE A-1. PRECIPITATION ZONES

Zone	Location
1	West of the Rio Grande
2	Between the Rio Grande and San Mateo
3	Between San Mateo and Eubank, North of Interstate 40; and between San Mateo and the East boundary of Range 4 East, South of Interstate 40
4	East of Eubank, North of Interstate 40; and East of the East boundary of Range 4 East, South of Interstate 40

FIGURE A-1



Where a watershed extends across a zone boundary, use the zone which contains the largest portion of the watershed.

EXAMPLE A-4
Find the 100-year 24-hour and 4-day runoff volume, V_{1440} and V_{4days}, for the area in Example A-3.
V_{360} = 2.41 acre-feet
V_{1440} = 2.41 + 7 ac * (2.66 - 2.20) / 12 = 2.68 acre-feet
V_{4DAYS} = 2.41 + 7 ac * (3.12 - 2.20) / 12 = 2.95 acre-feet

A.6 PEAK DISCHARGE RATE FOR SMALL WATERSHEDS

Small watersheds are less than or equal to 40 acres.

Peak Discharge

Using a 0.2-hour (12-minute) time of concentration, peak discharge, Q_P, per acre is the volume of excess precipitation in the heaviest 12-minute portion of the storm, divided by the time increment 12 minutes, and multiplied by an attenuation factor. The attenuation factor (0.59 for treatment A, 0.67 for treatment B, 0.75 for treatment C and 0.93 for treatment D) describes the effect of routing. Determine the peak discharge using the values in TABLE A-9, which have been adjusted to consider the effects of initial abstraction.

TABLE A-9. PEAK DISCHARGE (CFS/ACRE)

Zone	Treatment 100-YR [2-YR, 10-YR]			
	A	B	C	D
1	1.29 [0.00, 0.24]	2.03 [0.03, 0.76]	2.87 [0.47, 1.49]	4.37 [1.69, 2.89]
2	1.56 [0.00, 0.38]	2.28 [0.08, 0.95]	3.14 [0.60, 1.71]	4.70 [1.86, 3.14]
3	1.87 [0.00, 0.58]	2.60 [0.21,1.19]	3.45 [0.78, 2.00]	5.02 [2.04, 3.39]
4	2.20 [0.05, 0.87]	2.92 [0.38, 1.45]	3.73 [1.00, 2.26]	5.25 [2.17, 3.57]

TABLE A-4. LAND TREATMENTS	
Treatment	**Land Condition**
A	Soil uncompacted by human activity with 0 to 10 percent slopes. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, groundcover and infiltration capacity.
B	Irrigated lawns, parks and golf courses with 0 to 10 percent slopes. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes greater than 10 percent and less than 20 percent.
C	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Gravel or rock on plastic (desert landscaping). Irrigated lawns and parks with slopes greater than 10 percent. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes at 20 percent or greater. Native grass, weed and shrub areas with clay or clay loam soils and other soils of very low permeability as classified by SCS Hydrologic Soil Group D.
D	Impervious areas, pavement and roofs.
Most watersheds contain a mix of land treatments. To determine proportional treatments, measure respective subareas. In lieu of specific measurement for treatment D, the areal percentages in TABLE A-5 may be employed.	

To determine the peak rate of discharge,

1) Determine the area in each treatment, A_A, A_B, A_C and A_D.

2) Multiply the peak rate for each treatment by the respective areas and sum to compute the total Q_P.

$$\text{Total } Q_P = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D \qquad \text{(a-10)}$$

Example A-5

Find 100-year Q_P for 14 acres in zone 1. The four land treatments are: 3 acres in treatment A, 5 acres in treatment B, 2 acres in treatment C and 4 acres in treatment D.

Total Q_P = (1.29 * 3) + (2.03 * 5) + (2.87 * 2) + (4.37 * 4) = 37.24 cfs

3) Approximately the same results can be achieved by a Rational Method solution. The 0.2-hour (12-minute) peak intensities, I, are given in TABLE A-10 and Rational Method coefficients, C, are given in TABLE A-11.

$$\text{Total } Q_P = (C_A * I * A_A) + (C_B * I * A_B) + (C_C * I * A_C) + (C_D * I * A_D) \qquad \text{(a-11)}$$

TABLE A-10. PEAK INTENSITY (IN/HR at t_c = 0.2 hour)

Zone	Intensity 100-YR [2-YR, 10-YR]
1	4.70 [1.84, 3.14]
2	5.05 [2.04, 3.41]
3	5.38 [2.21, 3.65]
4	5.61 [2.34, 3.83]

TABLE A-6. INITIAL ABSTRACTION (IA)	
Treatment	**Initial Abstraction (inches)**
A	0.65
B	0.50
C	0.35
D	0.10

Infiltration is the only significant abstraction after the initial abstraction. After initial abstraction is satisfied, treat infiltration as a constant loss rate as specified in TABLE A-7.

TABLE A-7. INFILTRATION (INF)	
Treatment	**Loss Rate (inches/hour)**
A	1.67
B	1.25
C	0.83
D	0.04*
* Treatment D infiltration rate is applicable from 0 to 3 hours; use uniform reduction from 3 to 6 hours, with no infiltration after 6 hours.	

Runoff from a previous event can saturate a channel bed, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

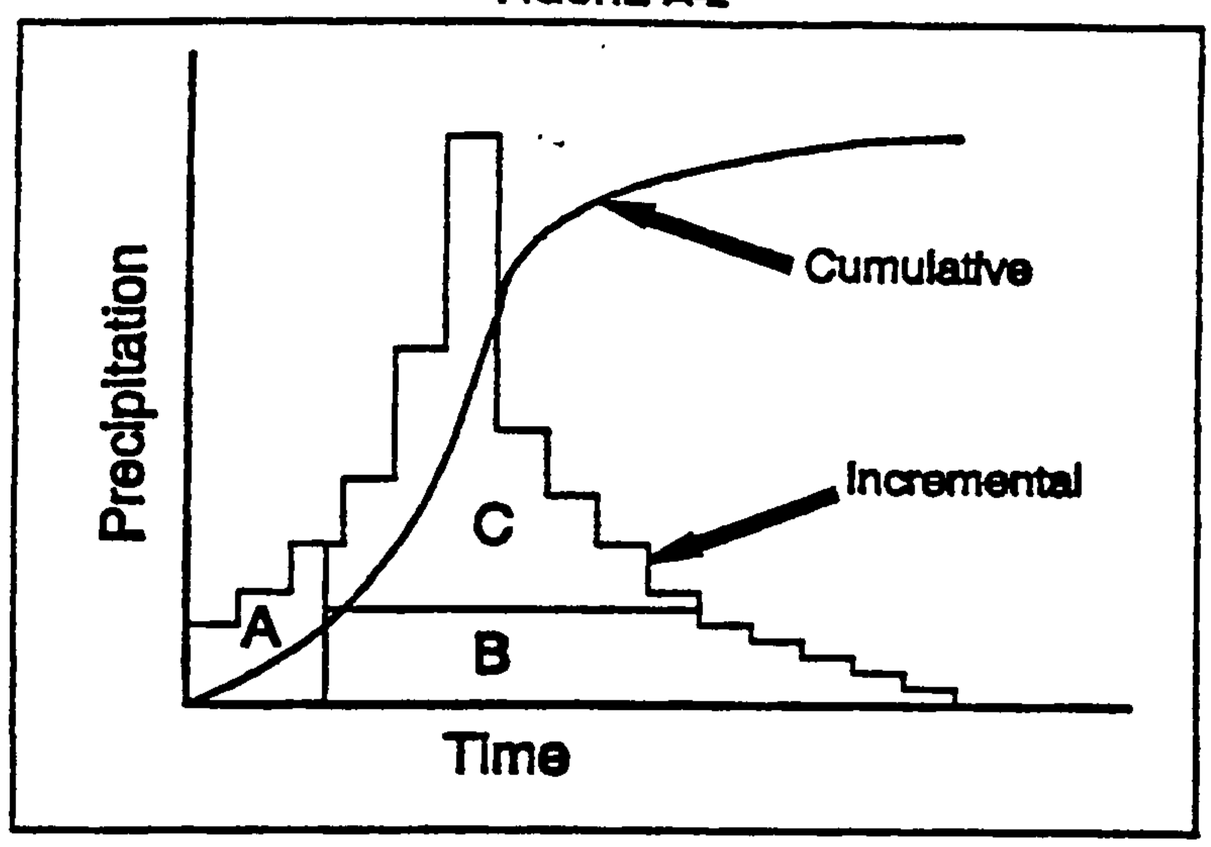
USED FOR POND SIZING

A.5 EXCESS PRECIPITATION & VOLUMETRIC RUNOFF

Excess precipitation, E, is the depth of precipitation remaining after abstractions are removed. Excess precipitation does not depend on watershed area. Excess precipitation is determined by subtracting the initial abstraction and infiltration from the design storm hydrograph. FIGURE A-2 illustrates the development of excess precipitation. The curved line plots cumulative precipitation. Precipitation intensities (in/hr) are shown as a histogram. Initial abstraction is area A. The horizontal line is at a height corresponding to the infiltration rate. Infiltration loss is area B. The remaining histogram, area C, is excess precipitation.

"POND SIZING"

FIGURE A-2



Excess precipitation, E, by zone and treatment is summarized in TABLE A-8.

(NOTE: In this table and several tables which follow, corresponding values for 2- and 10- year storms are shown in brackets below each 100-year value)

TABLE A-8: EXCESS PRECIPITATION, E (INCHES) - 6 HOUR STORM

Zone	Treatment 100-YR [2-YR, 10-YR]			
	A	B	C	D
1	0.44 [0.00, 0.08]	0.67 [0.01, 0.22]	0.99 [0.12, 0.44]	1.97 [0.72, 1.24]
2	0.53 [0.00, 0.13]	0.78 [0.02, 0.28]	1.13 [0.15, 0.52]	2.12 [0.79, 1.34]
3	0.66 [0.00, 0.19]	0.92 [0.06, 0.36]	1.29 [0.20, 0.62]	2.36 [0.89, 1.50]

"Pond Sizing"

To determine the volume of runoff,

1) Determine the area in each treatment, A_A, A_B, A_C, A_D

2) Compute the weighted excess precipitation, E

$$\text{Weighted E} = \frac{E_A A_A + E_B A_B + E_C A_C + E_D A_D}{A_A + A_B + A_C + A_D} \qquad \text{(a-5)}$$

3) Multiply the weighted E by the watershed area.

$$V_{360} \text{ (as volume)} = \text{weighted E} * (A_A + A_B + A_C + A_D) \qquad \text{(a-6)}$$

EXAMPLE A-3
Find the 100-year V_{360} for 30 acres in zone 1. Eight acres are treatment A, 10 acres are treatment B, 5 acres are treatment C, and 7 acres are treatment D.
Weighted E = ((8 * 0.44) + (10 * 0.67) + (5 * 0.99) + (7 * 1.97)) / 30 = 0.965 inches
Volume = (0.965 * 30) / 12 = 2.41 acre-ft. = V_{360}

For ponds which hold water for longer than 6 hours, longer duration storms are required to establish runoff volumes. Since the additional precipitation is assumed to occur over a long period, the additional volume is based on the runoff from the impervious areas only.

For 24-hour storms:

$$V_{1440} = V_{360} + A_D * (P_{1440} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-7)}$$

For 4-day storms:

$$V_{4DAYS} = V_{360} + A_D * (P_{4DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-8)}$$

STORM FOR RETENTION POND SIZING

For 10-day storms:

$$V_{10DAYS} = V_{360} + A_D * (P_{10DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-9)}$$

"POND SIZING"

"Type B" basins are generally placed downstream of and/or in conjunction with "Type A" basins on streets other than arterials and collectors. This type basin has potential to collect substantial runoff when the grating is clean. If "Type B" basins are used alone, without a "Type A" within 150 feet upstream, the capacity shown in Plate 22.3 D-7 should be reduced 15% due to clogging. "Type B" basins are used with standard 8" curb and gutter.

"Type C" basins are generally placed downstream of and/or in conjunction with "Type A" basins. If "Type C" basins are used without a "Type A" within 150 feet upstream, the capacity shown in Plates 22.3 D-5 and 22.3 D-6 should be reduced 15% for clogging. "Type C" basins are used with standard 8" curb and gutter.

"Type D" basins are generally used on streets with slope greater than 5%, in driveways and in certain valley areas as described above. "Type D" basins can be used with either standard 8" curb and gutter or with mountable curb.

The number of catch basins to be connected in series should not exceed two. If the connection of more than two catch basins in series is unavoidable, consideration should be given to designing a lateral drain.

c. Connector Pipe
   (1) The minimum diameter of connector pipe is 18 inches.
   (2) The horizontal alignment of connector pipes must not contain angle points or bends, unless approved by the City Engineer.
   (3) Connector pipes outletting into a pipe from both sides of a street should be offset 8 feet or more at the main line. Exceptions to this criterion must be approved by the City Engineer. Connections at manholes or junction structures are preferred.
   (4) ~~The minimum length of connector pipe to be installed between catch basins in series shall be 12 feet, unless otherwise approved by the City Engineer.~~ The catch basin spacing shall be a minimum of 25 feet distance between curb transitions.
   (5) Catch basin connector pipes shall outlet at the downstream end of the catch basins, unless prevented by field conditions. Downstream, in this paragraph, refers to the directions of the gutter slope at the catch basin in question.
   (6) Where feasible, connector pipes should be located so as to avoid, as much as possible, cutting into existing cross gutters and spandrels.
   (7) The conversions of 'type A's, B's or C's to D's basins will not be permitted. If the catch basin is in conflict with a driveway, the catch basin will be removed and replaced with another inlet outside of the driveway. To avoid conflicts with driveways, the engineer should identify the proposed driveways on the grading plan when catch basins front the lots.

E. <u>Street Hydraulics</u>
   1. A secondary use of the street network is the conveyance of storm runoff. This secondary use must always be subsidiary to the primary function of streets which is the safe conveyance of people and vehicles. The goals of street hydraulic design are therefore:
      a. To provide an economical means of transporting storm runoff.

      b. To ensure that the safety and convenience of the public are preserved.
      c. To prevent storm runoff, once collected by the street system, from leaving the street right-of-way except at specially designated locations.

   2. Street hydraulic design critical are as follows:

a. Manning's roughness coefficient is 0.017.
b. Flow depths plus velocity head ($v^2/2g$)in the event of the 100-year design discharge may not exceed 0.2 feet above curb height ~~or 0.87 feet at any location and shall be contained within the street right-of-way~~.
c. Flow depths in the event of the 10-year design discharge may not exceed 0.5 feet in any collector or arterial street. One ~~twelve (12) foot driving~~ lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.
d. The product of depth times velocity shall not exceed 6.5 in any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet.)

3. For streets with more than two driving lanes in each direction:

   a. The product of depth times velocity may not exceed 6.5 at any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet).
   b. Inverted crown streets are prohibited.
   c. The assumption of equal flow distribution between gutters on undivided streets and between street sections on divided streets is only valid where its validity can be demonstrated.

4. Plates 22.3 D-1 through 22.3 D-4 may be used where applicable in the hydraulic design of streets. T-intersections, radical slope changes and intersections are potential locations for hydraulic jumps when upstream slopes are steeper than critical slope.

   When conditions indicate that a hydraulic jump or that the effects of superelevation will allow runoff to exceed street hydraulic design criteria, provisions must be made to for treatment of the problem. The warping of street sections and the construction of deflector walls for these purposes is prohibited unless specifically authorized by the City Engineer.

5. Intersections and other radical changes in street cross section and slope require special consideration whenever the flow depth/street slope relationship results in flows occurring in the supercritical flow regime. The critical slope line shown on the street rating curves is used to determine on which side of critical depth the flow occurs and if slope or cross section changes will allow the flow to cross through critical depth from supercritical.

   If flow is likely to cross into the subcritical flow range, then Plate 22.3 E-1, "Tail Water vs. Froude Number" is used to determine the height and Plate 22.3 E-2, "Length of Jump vs. Froude Number" figure is used to determine jump length. The height of jump should not exceed 0.2 feet above curb height ~~0.87 feet~~ and ~~the jump should~~ shall be contained within the street right-of-way.

APPENDIX B

PLATES

PLATE 1 – DRAINAGE BASIN MAP
PLATE 2 – DRAINAGE BASIN MAP

APPENDIX C

CALCULATIONS

System 1 – Sta: 161+90.78 to 164+86.89

System 2 – Sta: 173+72.05 to 183+50

System 3 – Sta: 199+38.47 to 206+09.82 on Eubank
Sta: 0+00 to 8+71.97 on Southern

System 4 – Sta: 210+25.83 to 220+52.97

SYSTEM 1 HYDROLOGY CALCS:

- BASIN #2 (WEST SIDE):

A = 1.82 ACRES

100% "D"

Q = 1.82 * 5.02 = 9.1 cfs

- BASIN #1 (EAST SIDE):

A = 3.64 ACRES

90% "D"

10% "C"

Q = (0.9 * 3.64 * 5.02) + (0.1 * 3.64 * 3.45) = 17.7 cfs

SYSTEM 2

"SYSTEM 2" DRAINAGE BASIN CALCULATIONS:

- **BASIN 3**

AREA = 3.15 ACRES

LAND TREATMENT TYPES: 100% "D"

$Q_{100\text{-}YR}$ = 3.15 * 5.02 = 15.8 cfs

- **BASIN 4**

AREA = 3.21 ACRES

LAND TREATMENT TYPES: 100% "D"

$Q_{100\text{-}YR}$ = 3.21 * 5.02 = 16.1 cfs

"System 3" Drainage Basin Calculations

Basin 5

Area = 2.84 acres
Land treatment types: 100% "D"

$Q_{100\text{-}yr} = 2.84 * 5.02 = \underline{14.3 \text{ cfs}}$

Basin 5 Runoff Volume

Excess precipitation $= E = \dfrac{2.84 * 2.36}{2.84} = 2.36$ inches

$V_{360} = \dfrac{2.36}{12} * (2.84 * 43{,}560) = 24{,}330 \text{ ft}^3 = 0.56$ acre-feet

$V_{10\text{-days}} = 0.56 + [2.84 * (4.90 - 2.60)/12] = 1.10$ acre-feet $= 48{,}040 \text{ ft}^3$

Basin 6

Area = 2.66 acres
Land treatment types: 100% "D"

$Q_{100\text{-}yr} = 2.66 * 5.02 = \underline{13.4 \text{ cfs}}$

Basin 5 Runoff Volume:

Excess precipitation $= E = 2.36$ inches

$V_{360} = \dfrac{2.36}{12} * (2.66 * 43{,}560) = 22{,}790 \text{ ft}^3 = 0.52$ acre-feet

$V_{10\text{-days}} = 0.52 + [2.66 * (4.90 - 2.60)/12] = 1.03$ acre-feet $= 44{,}870 \text{ ft}^3$

SYSTEM 3

· APS PROPERTY (UNDEVELOPED)

AREA = 13.3 ACRES

LAND TREATMENT TYPES : 85% "A"
15% "C"

$Q_{100\text{-}YR} = (0.85 * 13.3 * 1.87) + (0.15 * 13.3 * 3.45) = \underline{28.0 \text{ cfs}}$

· APS PROPERTY (DEVELOPED)

LAND TREATMENT TYPES : 25% "B"
25% "C"
50% "D"

$Q_{100\text{-}YR} = (0.25 * 13.3 * 2.6) + (0.25 * 13.3 * 3.45) + (0.5 * 13.3 * 5.02) = 53.5$ CFS

USE 55 CFS

· APS PROPERTY RUNOFF VOLUME (UNDEVELOPED)

EXCESS PRECIPITATION = E

$$\text{WEIGHTED } E = \frac{(13.3 * 0.85 * 0.66) + (13.3 * 0.15 * 1.29)}{13.3} = 0.755 \text{ INCHES}$$

$$V_{360} = \frac{0.755}{12} * (13.3 * 43{,}560) = 36{,}450 \text{ FT}^3 = 0.84 \text{ ACRE-FEET}$$

$V_{360} = V_{10\text{-}DAYS}$ BECAUSE THE UNDEVELOPED APS PROPERTY HAS NO LAND TREATMENT OF TYPE "D".

Bohannan ▲ Huston

PROJECT NAME _____ SHEET _____ OF _____

PROJECT NO. _____ BY _____ DATE _____

SUBJECT System 3 CH'D _____ DATE _____

SYSTEM 3

"System 3" Pond Sizing

Assumptions: · APS property undeveloped (13.3 acres)
· Eubank roadway fully developed (5.5 acres)

Treatment types:

· APS: 85% "A"
15% "C"

· Eubank Roadway: 100% "D"

Excess precipitation = E

$$\text{Weighted } E = \frac{(0.66 * 13.3 * 0.85) + (1.29 * 13.3 * 0.15) + (2.36 * 5.5)}{18.8}$$

= 1.22 inches of excess precipitation

$$V_{360} = \frac{1.22}{12} * (18.8 \times 43{,}560) = 83{,}260\ ft^3 = 1.91 \text{ acre feet}$$

$$V_{10\text{-days}} = 1.91 + [5.5 * (4.90 - 2.60)/12] = 2.97 \text{ acre-feet} = \underline{\underline{129{,}375\ ft^3}}$$

Temporary retention pond capacity use 129,500 ft³

Bohannan Huston

PROJECT NAME _____ SHEET _____ OF _____
PROJECT NO _____ BY _____ DATE _____
SUBJECT System 3 _____ CH'D _____ DATE _____

Capacity of ported riser

Eubank Expansion - APS POND
PORTED RCP RISER TOWER DESIGN

DESIGN PARAMATERS
Peak Discharge Rate at 100 year water surface =
Peak Discharge Rate at PMP water surface =
Outlet Works to be ported riser tower
Riser tower invert =
Top of Pond = 5069.45
Orifice Coefficient = 0.61

2 rows of 6, 6" openings
rows 1' apart, center-to-center

36" Riser

PORTED RCP RISER WITH TOP OPENING

	WATER ELEVATION (ft)	OPENING ROW NUMBER	NUMBER OF OPENINGS	OPENING DIAMETER (in)	OPENING AREA (sf)	ROW #1 OPENING HEAD (ft)	ROW #2 OPENING HEAD (ft)	ROW #3 OPENING HEAD (ft)	ROW #4 OPENING HEAD (ft)	ROW #5 OPENING HEAD (ft)	CUM. PORT FLOW RATE (cfs)	TOWER TOP OPENING WEIR LENGTH (ft)	WEIR HEAD (ft)	WEIR FLOW RATE (cfs)	TOTAL FLOW RATE (cfs)	ADJUSTED FLOW RATE (cfs)
											0.00			0.00	0.00	0.00
POND INVERT	5466	1	6	6	1.18	0.00	0.00	0.00	0.00	0.00	0.00			0.00	0.00	0 00
	5467	2	6	6	1.18	0.50	0.00	0 00	0.00	0.00	4.08			0 00	4.08	2.04
TOP OF TOWER	5468	3	0	4	0.00	1.50	0.50	0.00	0.00	0 00	11.14	9.42	0	0.00	11 14	5 57
	5469		0	8	0 00	2.50	1.50	1.00	0.00	0.00	16.18		1	24.77	40.96	20.48
	5469		0	8	0.00	2 50	1.50	1.00	1.00	0.00	16.18		1	24.77	40.96	20.48
TOP OF POND	5469.45					2.95	1 95	1.45	1.45	0.45	17.96		1.45	43.26	61 22	30 61

28.27433

30.61 > 28 CFS ✓

Capacity of outflow pipe for APS Pond/Ported Riser

"Calculated Values"

Pipes

Orifice diameter (in.)	=	30									
Area (ft^2)	=	4.91									
C	=	0.6									
Water surface elevation (ft)	=	0	1	2	3	4	5	5.24			
Headwater (ft)	=	0.00	0.00	0.75	1.75	2.75	3.75	3.99	0.00	0.00	0.00
Q (ft^3/s)	=	0.0	0.0	20.5	31.3	39.2	45.8	47.2	0.0	0.0	0.0
Number of Barrells	1	0.0	0.0	20.5	31.3	39.2	45.8	47.2	0.0	0.0	0.0

47.2 > 28.0 CFS ✓

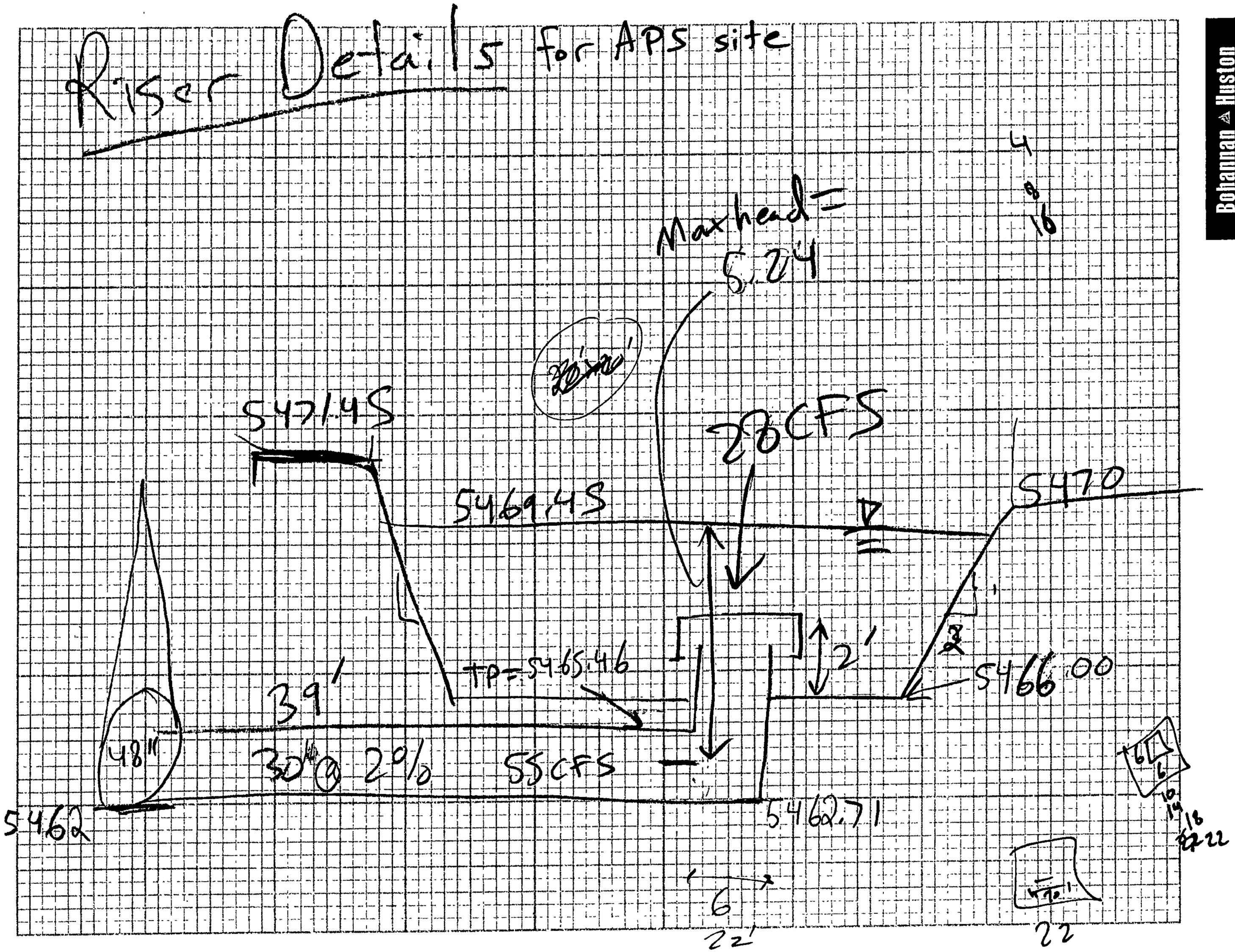
Capacity of 30" = 62.39 cfs > 55 cfs ✓

with n = .013

s = 2%

admin/hydro/programs/Orificeflow.xls[Orifice flow]6/4/2002

Riser Details for APS site



Bohannan Huston

ENGINEERS PLANNERS PHOTOGRAMMETRISTS
SURVEYORS SOFTWARE DEVELOPERS

PROJECT NAME _____ SHEET _____ OF _____

PROJECT NO. _____ BY _____ DATE _____

SUBJECT _____ CH'D _____ DATE _____

Pond @ Eubank and Southern

Infiltration (See Attached)

$= 3 \times 10^{-6}$ ft/s * 96 hrs = 1.04'

Evaporation = 7'/year (from State engineer off.)

= .08' in 96 hrs ≈ 1"

Total = 1.12'/96 hrs ≈ 13"/96 hrs

2yr - 24hr storm produces approx. 3' deep accumulation in pond.

Pond is 8' deep.

Bohannan Huston

ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

PROJECT NAME Eubank - Pond SHEET ____ OF ____

PROJECT NO. 00268 BY GMM DATE 6/19/02

SUBJECT Pond Calculations CH'D ____ DATE ____

Runoff volume in pond resulting from a 2-yr 24-hr storm:

HYDROGRAPH FROM AREA PONDIN

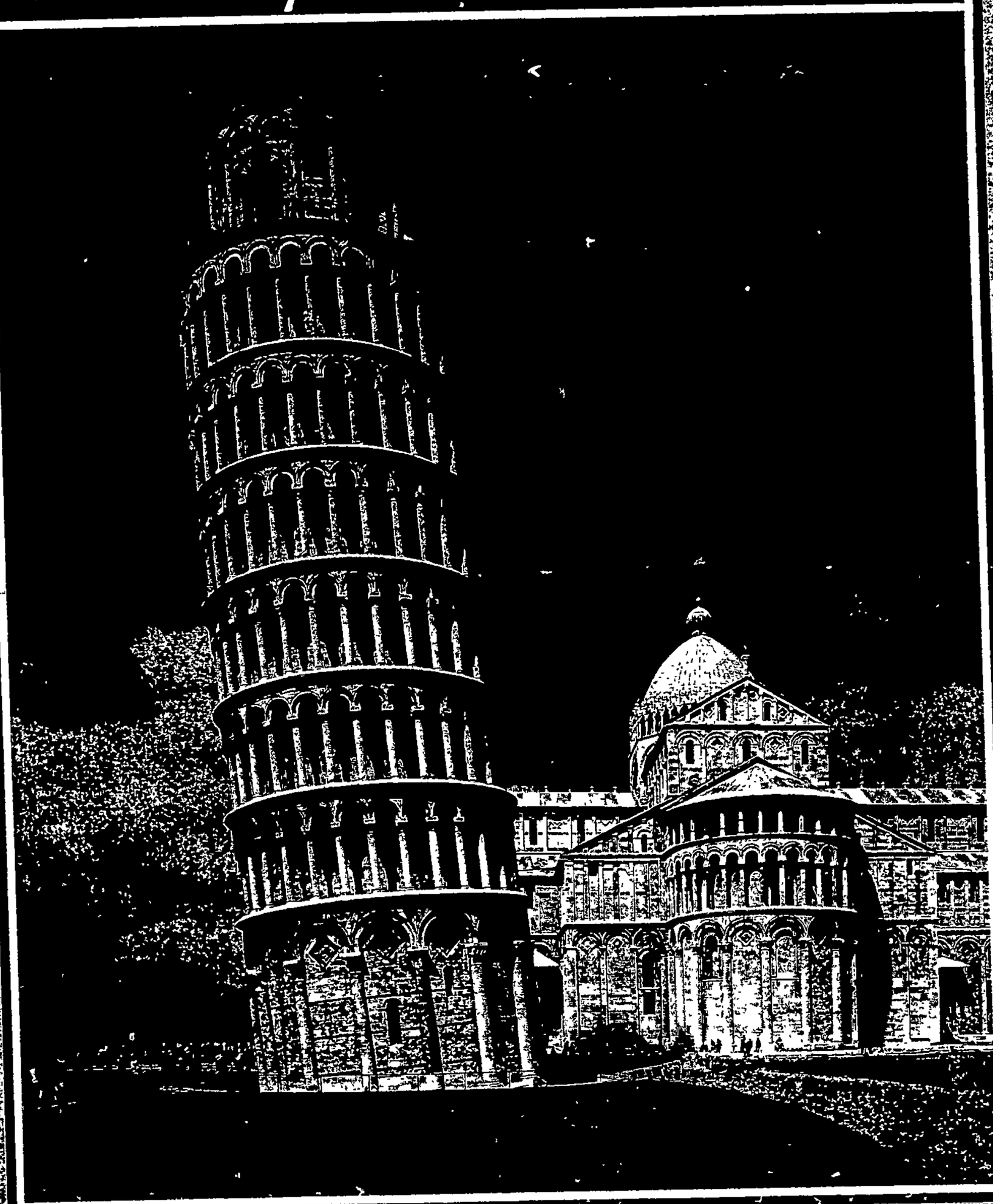
TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS
.000	.0	5.500	.1	11.000	.1	16.500	.1
.500	.0	6.000	.1	11.500	.1	17.000	.1
1.000	.0	6.500	.1	12.000	.1	17.500	.1
1.500	10.7	7.000	.1	12.500	.1	18.000	.1
2.000	3.0	7.500	.1	13.000	.1	18.500	.1
2.500	.4	8.000	.1	13.500	.1	19.000	.1
3.000	.1	8.500	.1	14.000	.1	19.500	.1
3.500	.0	9.000	.1	14.500	.1	20.000	.1
4.000	.0	9.500	.1	15.000	.1	20.500	.1
4.500	.0	10.000	.1	15.500	.1	21.000	.1
5.000	.1	10.500	.1	16.000	.1	21.500	.1

RUNOFF VOLUME = .33713 INCHES=.5286 ACRE-FEET=**23026 cu. ft.**

PEAK DISCHARGE RATE = 11.35 CFS AT 1.550 HOURS BASIN AREA = .0294 SQ. MI.

GEOTECHNICAL ENGINEERING

Principles and Practices



DONALD P. CODUTO

Q also will be 1,000,000 times smaller. The low k in clays is due to the small particle size (and therefore small void size). It is not due to water being absorbed by the clay.

TABLE 7.1 TYPICAL VALUES OF HYDRAULIC CONDUCTIVITY, k, FOR SATURATED SOILS

Soil Description	Hydraulic Conductivity, k	
	(cm/s)	(ft/s)
Clean gravel	1 to 100	3×10^{-2} to 3
Sand-gravel mixtures	10^{-2} to 10	3×10^{-4} to 0.3
Clean coarse sand	10^{-2} to 1	3×10^{-4} to 3×10^{-2}
Fine sand	10^{-3} to 10^{-1}	3×10^{-5} to 3×10^{-3}
Silty sand	10^{-3} to 10^{-2}	3×10^{-5} to 3×10^{-4}
Clayey sand	10^{-4} to 10^{-2}	3×10^{-6} to 3×10^{-4}
Silt	10^{-8} to 10^{-3}	3×10^{-10} to 3×10^{-5}
Clay	10^{-10} to 10^{-6}	3×10^{-12} to 3×10^{-8}

Geotechnical engineers also use the term *coefficient of permeability* to describe k. Unfortunately, this term can generate some confusion because it has at least two definitions:

- The *coefficient of permeability*, k, in Darcy's Law describes the ease with which a certain liquid flows through a certain soil. It depends on both the soil and the liquid, has units of L/t, and is the same as *hydraulic conductivity*.
- The *intrinsic permeability* (also called the *specific permeability*) which depends only on the soil. It has units of L^2. This definition is used primarily by hydrogeologists and petroleum geologists.

Both parameters are frequently called "permeability" and the variables K and k have been used for both. In addition, the intrinsic permeability is sometimes measured with a special unit called Darcys, even though it is not the proper parameter for use in Equation 7.8. These inconsistencies in terminology can be a source of confusion. Whenever in doubt, check the units to determine which "permeability" is being used.

To avoid confusion, geotechnical engineers are gradually dropping the term "coefficient of permeability" and using "hydraulic conductivity" instead.

Hydraulic Conductivity Tests (Permeability Tests)

Various laboratory and in-situ tests are available to measure the hydraulic conductivity. However, the results from both types are often judged with some skepticism because we are not sure if the test samples are truly representative. Even small differences in the soil classification can make a big difference in k, as illustrated in Table 7.1. Thus, it is good to perform many tests and review the scatter in the results.

Anc
represent
tests are b
Eve
order of =
significan
conductiv

Co

The *cons*
head of w

Figure 7.11 A co
constant heads. Th

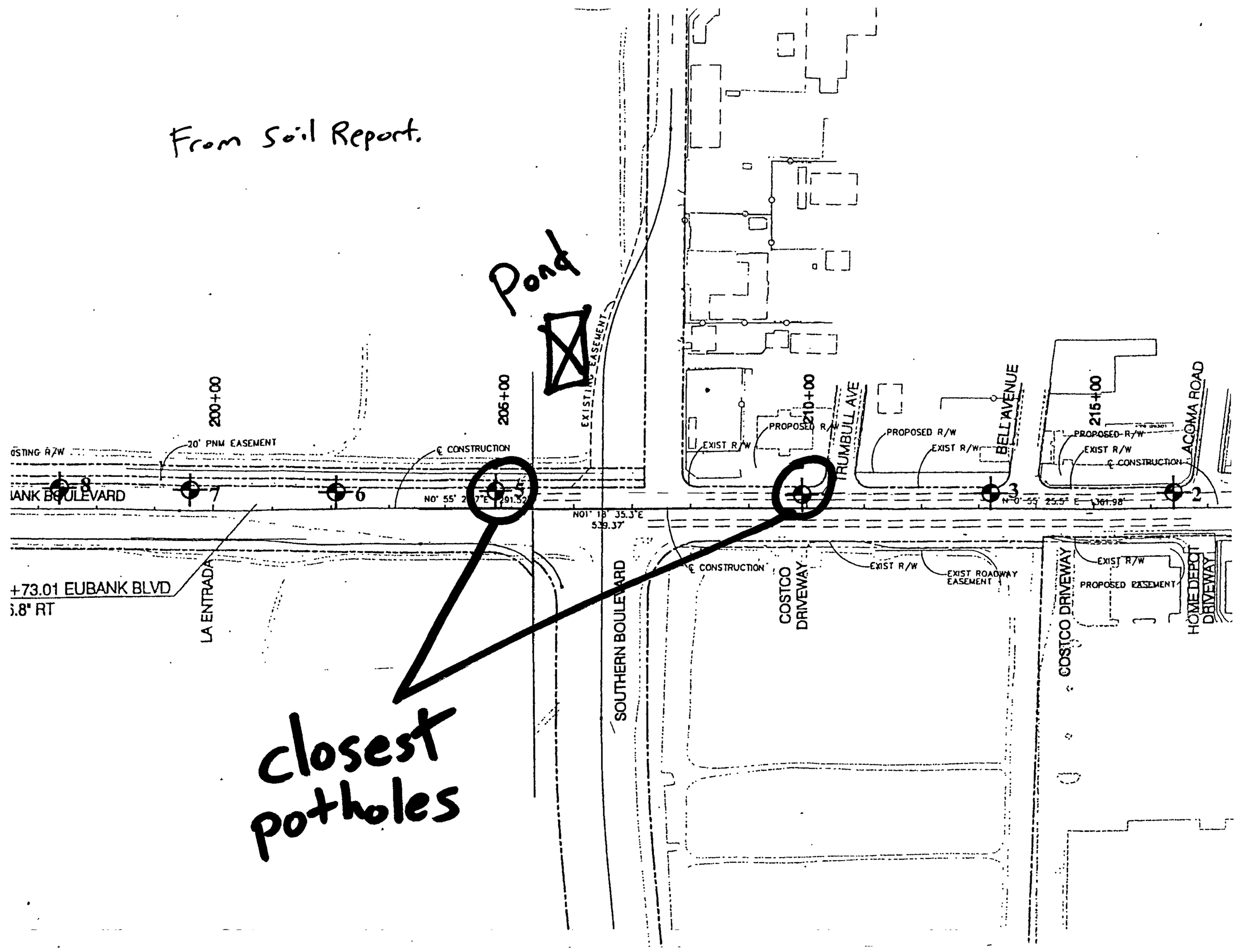
Pond depth resulting from a 2- yr 24-hr storm:

Stage Storage Calculations (Average End Area Method)

Pond Title
Job No. 2 year storm requires 23026 cu. Ft. (reaches depth of approx. 2.7 ft

ELEV. (FT)	AREA (AC)	AVG. AREA (AC)	HEIGHT (FT)	VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)	TOTAL VOLUME (CU-FT)
2907	5453	0.066736			0.0	0
		0.116793	1	0.1		0
7268	5454	0.16685			0.1	5087.5
		0.212121	1	0.2		0
11212	5455	0.257392			0.3	14327.5
		0.280108	1	0.3		0
13191	5456	0.302824			0.6	26529
		0.326194	1	0.3		0
15227	5457	0.349564			0.9	40738
		0.373577	1	0.4		0
17319	5458	0.39759			1.3	57011
		0.422257	1	0.4		0
19468	5459	0.446924			1.7	75404.5
		0.472234	1	0.5		0
21673	5460	0.497544			2.2	95975
		0.523508	1	0.5		0
23935	5461	0.549472			2.7	118779
		0.57609	1	0.6		0
26254	5462	0.602709			[illegible]	143873.5
		0.600023	1	0.6		0
26020	5463	0.597337			3.9	170010.5
		0.529706	1	0.5		0
20128	5464	0.462075			4.4	193084.5
		0.401653	1	0.4		0
14864	5465	0.34123			4.8	210580.5
		0.275253	1	0.3		0
9116	5466	0.209275			5.1	222570.5
		0.143974	1	0.1		0
3427	5467	0.078673			5.3	228842

From Soil Report.



V & A

LOG OF TEST HOLE NO. 4

From soil report

Project: Reconstruction of Eubank Boulevard
Project No.: 01-1-259
Elevation - Top of Test Hole: N/A
Date Drilled: 8/4/2001
Depth to Groundwater: Not Encountered
Method: 6" HSA

Depth, feet	Blows/Foot	Sample Type	Dry Density pcf	Water Content, %	Additional Testing	Unified Classification	Material Description
							8" asphalt
1						SM	SAND, very silty, fine to coarse grained, gravelly, medium dense, medium moist, light brown
2	29	S		3.5	1		
3							
4						GM	GRAVEL, silty, very sandy, fine to coarse grained, medium dense, medium moist, light brown to grey
5							
6	5	S		2.3	1	SM	SAND, silty, fine to coarse grained, gravelly, loose, moist, brown
7							
8							
9						CL	CLAY, sandy, fine grained, medium stiff, medium moist, brown
10							
11	8	S		8.2			
12							Bottom of hole at 11½'

ADDITIONAL TESTS: 1= Sieve Analysis 2= Atterberg Limits 3=Direct Shear 4=R-Value 5=Other

Figure: 5

V & A

LOG OF TEST HOLE NO. 5

From Soil Report

Project: Reconstruction of Eubank Boulevard
Elevation - Top of Test Hole: N/A
Depth to Groundwater: Not Encountered

Project No.: 01-1-259
Date Drilled: 8/4/2001
Method: 6" HSA

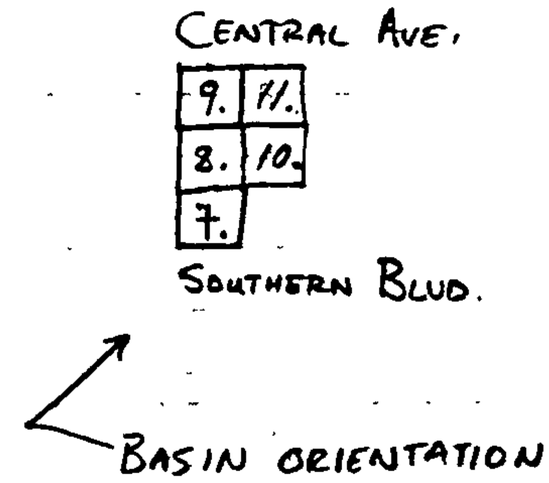
Depth, feet	Blows/Foot	Sample Type	Dry Density pcf	Water Content, %	Additional Testing	Unified Classification	Material Description
							8" asphalt
1						SM	SAND, very silty, fine to coarse grained, very gravelly, medium dense, moist, brown to light brown, with thin lenses of clay
2	21	S		7.5	1		
3							
4							
5							Silty, dense, medium moist, brown, no clay
6	35	S		1.9			
7							
8							
9							
10							
11	8	S		6.5			Very silty, fine to medium grained, trace gravel, loose, moist
12							Bottom of hole at 11½'

ADDITIONAL TESTS: 1= Sieve Analysis 2= Atterberg Limits 3=Direct Shear 4=R-Value 5=Other

Figure: 6

SYSTEM 4

- NORTH OF SOUTHERN BLVD.



- BASIN 7: AREA = 0.614 ACRES

  $Q = (0.614 * 5.02) \simeq \underline{3.1 \text{ cfs}}$ ✓

- BASIN 8: A = 0.429 ACRES

  $Q = (0.429 * 5.02) \simeq \underline{2.2 \text{ cfs}}$ ✓

- BASIN 9: AREA = 1.378 ACRES

  $Q = (1.378 * 5.02) \simeq \underline{6.9 \text{ cfs}}$ ✓

- BASIN 10: AREA 2.300 ACRES

  $Q = (0.287 * 5.02) \simeq \underline{11.5 \text{ cfs}}$ ✓

- BASIN 11: AREA = 1.184 ACRES

  $Q = (1.184 * 5.02) \simeq \underline{6.0 \text{ cfs}}$

- 100-YR DESIGN STORM
- ZONE 3

- BASIN 10 AND 11 CONTRIBUTE FLOWS ENTERING CATCH BASINS AT SOUTHERN "HOME DEPOT" ENTRANCE.

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .005

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.17	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.28	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.36	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.44	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.51	0.78	0.00	0.05
0.06	0.06	0.03	0.0	0.99	0.58	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.64	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.70	1.25	0.01	0.09
0.09	0.09	0.06	0.0	1.48	0.76	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.81	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	0.86	1.72	0.01	0.12
0.12	0.12	0.11	0.1	1.97	0.92	1.88	0.01	0.13
0.13	0.13	0.13	0.1	2.14	0.97	2.03	0.01	0.14
0.14	0.14	0.16	0.1	2.66	0.93	2.54	0.01	0.15
0.15	0.15	0.18	0.2	3.18	0.92	3.06	0.01	0.16
0.16	0.16	0.22	0.2	3.70	0.93	3.57	0.01	0.17
0.17	0.17	0.25	0.2	4.22	0.95	4.08	0.01	0.18
0.18	0.18	0.30	0.3	4.73	0.98	4.59	0.01	0.19
0.19	0.19	0.35	0.3	5.25	1.01	5.10	0.02	0.21
0.20	0.20	0.40	0.4	5.77	1.04	5.61	0.02	0.22
0.21	0.21	0.46	0.5	6.29	1.08	6.12	0.02	0.23
0.22	0.22	0.52	0.6	6.81	1.12	6.64	0.02	0.24
0.23	0.23	0.59	0.7	7.33	1.15	7.15	0.02	0.25
0.24	0.24	0.67	0.8	7.85	1.19	7.66	0.02	0.26
0.25	0.25	0.74	0.9	8.37	1.23	8.17	0.02	0.27
0.26	0.26	0.83	1.1	8.89	1.27	8.68	0.03	0.29
0.27	0.27	0.92	1.2	9.41	1.31	9.19	0.03	0.30
0.28	0.28	1.01	1.4	9.93	1.35	9.70	0.03	0.31
0.29	0.29	1.11	1.5	10.45	1.39	10.21	0.03	0.32
0.30	0.30	1.22	1.7	10.97	1.43	10.73	0.03	0.33
0.31	0.31	1.33	1.9	11.48	1.47	11.24	0.03	0.34
0.32	0.32	1.44	2.2	12.00	1.50	11.75	0.04	0.36
0.33	0.33	1.56	2.4	12.52	1.54	12.26	0.04	0.37
0.34	0.34	1.69	2.7	13.04	1.58	12.77	0.04	0.38
0.35	0.35	1.82	2.9	13.56	1.62	13.28	0.04	0.39
0.36	0.36	1.95	3.2	14.08	1.66	13.79	0.04	0.40
0.37	0.37	2.09	3.5	14.60	1.69	14.31	0.04	0.41
0.38	0.38	2.24	3.9	15.12	1.73	14.82	0.05	0.43
0.39	0.39	2.39	4.2	15.64	1.77	15.33	0.05	0.44
0.40	0.40	2.54	4.6	16.16	1.80	15.84	0.05	0.45
0.41	0.41	2.71	5.0	16.68	1.84	16.35	0.05	0.46
0.42	0.42	2.87	5.4	17.20	1.87	16.86	0.05	0.47
0.43	0.43	3.04	5.8	17.72	1.91	17.37	0.06	0.49
0.44	0.44	3.22	6.3	18.23	1.95	17.89	0.06	0.50
0.45	0.45	3.40	6.7	18.75	1.98	18.40	0.06	0.51

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	7.7	19.79	2.05	19.42	0.07	0.54
0.48	0.48	3.98	8.3	20.29	2.09	19.91	0.07	0.55
0.49	0.49	4.18	8.9	20.79	2.12	20.40	0.07	0.56
0.50	0.50	4.38	9.4	21.29	2.16	20.89	0.07	0.57
0.51	0.51	4.59	10.1	21.78	2.19	21.38	0.07	0.58

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0066

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.20	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.32	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.42	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.51	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.59	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.66	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.73	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.80	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	0.87	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.93	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	0.99	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.05	1.88	0.02	0.14
0.13	0.13	0.13	0.1	2.14	1.11	2.03	0.02	0.15
0.14	0.14	0.16	0.2	2.66	1.07	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.06	3.06	0.02	0.17
0.16	0.16	0.22	0.2	3.70	1.07	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.09	4.08	0.02	0.19
0.18	0.18	0.30	0.3	4.73	1.12	4.59	0.02	0.20
0.19	0.19	0.35	0.4	5.25	1.16	5.10	0.02	0.21
0.20	0.20	0.40	0.5	5.77	1.20	5.61	0.02	0.22
0.21	0.21	0.46	0.6	6.29	1.24	6.12	0.02	0.23
0.22	0.22	0.52	0.7	6.81	1.28	6.64	0.03	0.25
0.23	0.23	0.59	0.8	7.33	1.33	7.15	0.03	0.26
0.24	0.24	0.67	0.9	7.85	1.37	7.66	0.03	0.27
0.25	0.25	0.74	1.1	8.37	1.41	8.17	0.03	0.28
0.26	0.26	0.83	1.2	8.89	1.46	8.68	0.03	0.29
0.27	0.27	0.92	1.4	9.41	1.51	9.19	0.04	0.31
0.28	0.28	1.01	1.6	9.93	1.55	9.70	0.04	0.32
0.29	0.29	1.11	1.8	10.45	1.60	10.21	0.04	0.33
0.30	0.30	1.22	2.0	10.97	1.64	10.73	0.04	0.34
0.31	0.31	1.33	2.2	11.48	1.68	11.24	0.04	0.35
0.32	0.32	1.44	2.5	12.00	1.73	11.75	0.05	0.37
0.33	0.33	1.56	2.8	12.52	1.77	12.26	0.05	0.38
0.34	0.34	1.69	3.1	13.04	1.82	12.77	0.05	0.39
0.35	0.35	1.82	3.4	13.56	1.86	13.28	0.05	0.40
0.36	0.36	1.95	3.7	14.08	1.90	13.79	0.06	0.42
0.37	0.37	2.09	4.1	14.60	1.95	14.31	0.06	0.43
0.38	0.38	2.24	4.4	15.12	1.99	14.82	0.06	0.44
0.39	0.39	2.39	4.8	15.64	2.03	15.33	0.06	0.45
0.40	0.40	2.54	5.3	16.16	2.07	15.84	0.07	0.47
0.41	0.41	2.71	5.7	16.68	2.11	16.35	0.07	0.48
0.42	0.42	2.87	6.2	17.20	2.15	16.86	0.07	0.49
0.43	0.43	3.04	6.7	17.72	2.19	17.37	0.07	0.50
0.44	0.44	3.22	7.2	18.23	2.23	17.89	0.08	0.52
0.45	0.45	3.40	7.7	18.75	2.28	18.40	0.08	0.53

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	8.9	19.79	2.35	19.42	0.09	0.56
0.48	0.48	3.98	9.5	20.29	2.40	19.91	0.09	0.57
0.49	0.49	4.18	10.2	20.79	2.44	20.40	0.09	0.58
0.50	0.50	4.38	10.9	21.29	2.48	20.89	0.10	0.60
0.51	0.51	4.59	11.6	21.78	2.52	21.38	0.10	0.61

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .007

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.21	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.33	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.43	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.52	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.60	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.68	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.76	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.83	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	0.89	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.96	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	1.02	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.08	1.88	0.02	0.14
0.13	0.13	0.13	0.2	2.14	1.14	2.03	0.02	0.15
0.14	0.14	0.16	0.2	2.66	1.10	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.09	3.06	0.02	0.17
0.16	0.16	0.22	0.2	3.70	1.10	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.13	4.08	0.02	0.19
0.18	0.18	0.30	0.3	4.73	1.16	4.59	0.02	0.20
0.19	0.19	0.35	0.4	5.25	1.19	5.10	0.02	0.21
0.20	0.20	0.40	0.5	5.77	1.23	5.61	0.02	0.22
0.21	0.21	0.46	0.6	6.29	1.28	6.12	0.03	0.24
0.22	0.22	0.52	0.7	6.81	1.32	6.64	0.03	0.25
0.23	0.23	0.59	0.8	7.33	1.37	7.15	0.03	0.26
0.24	0.24	0.67	0.9	7.85	1.41	7.66	0.03	0.27
0.25	0.25	0.74	1.1	8.37	1.46	8.17	0.03	0.28
0.26	0.26	0.83	1.2	8.89	1.50	8.68	0.04	0.30
0.27	0.27	0.92	1.4	9.41	1.55	9.19	0.04	0.31
0.28	0.28	1.01	1.6	9.93	1.60	9.70	0.04	0.32
0.29	0.29	1.11	1.8	10.45	1.64	10.21	0.04	0.33
0.30	0.30	1.22	2.1	10.97	1.69	10.73	0.04	0.34
0.31	0.31	1.33	2.3	11.48	1.73	11.24	0.05	0.36
0.32	0.32	1.44	2.6	12.00	1.78	11.75	0.05	0.37
0.33	0.33	1.56	2.9	12.52	1.83	12.26	0.05	0.38
0.34	0.34	1.69	3.2	13.04	1.87	12.77	0.05	0.39
0.35	0.35	1.82	3.5	13.56	1.91	13.28	0.06	0.41
0.36	0.36	1.95	3.8	14.08	1.96	13.79	0.06	0.42
0.37	0.37	2.09	4.2	14.60	2.00	14.31	0.06	0.43
0.38	0.38	2.24	4.6	15.12	2.05	14.82	0.07	0.45
0.39	0.39	2.39	5.0	15.64	2.09	15.33	0.07	0.46
0.40	0.40	2.54	5.4	16.16	2.13	15.84	0.07	0.47
0.41	0.41	2.71	5.9	16.68	2.18	16.35	0.07	0.48
0.42	0.42	2.87	6.4	17.20	2.22	16.86	0.08	0.50
0.43	0.43	3.04	6.9	17.72	2.26	17.37	0.08	0.51
0.44	0.44	3.22	7.4	18.23	2.30	17.89	0.08	0.52
0.45	0.45	3.40	8.0	18.75	2.34	18.40	0.09	0.54

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	9.2	19.79	2.42	19.42	0.09	0.56
0.48	0.48	3.98	9.8	20.29	2.47	19.91	0.09	0.57
0.49	0.49	4.18	10.5	20.79	2.51	20.40	0.10	0.59
0.50	0.50	4.38	11.2	21.29	2.55	20.89	0.10	0.60
0.51	0.51	4.59	11.9	21.78	2.59	21.38	0.10	0.61

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0091

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.24	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.37	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.49	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.59	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.69	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.78	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.86	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.94	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	1.02	1.41	0.02	0.11
0.10	0.10	0.08	0.1	1.64	1.09	1.56	0.02	0.12
0.11	0.11	0.09	0.1	1.81	1.17	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.24	1.88	0.02	0.14
0.13	0.13	0.13	0.2	2.14	1.30	2.03	0.03	0.16
0.14	0.14	0.16	0.2	2.66	1.25	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.24	3.06	0.02	0.17
0.16	0.16	0.22	0.3	3.70	1.26	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.28	4.08	0.03	0.20
0.18	0.18	0.30	0.4	4.73	1.32	4.59	0.03	0.21
0.19	0.19	0.35	0.5	5.25	1.36	5.10	0.03	0.22
0.20	0.20	0.40	0.6	5.77	1.41	5.61	0.03	0.23
0.21	0.21	0.46	0.7	6.29	1.45	6.12	0.03	0.24
0.22	0.22	0.52	0.8	6.81	1.50	6.64	0.04	0.26
0.23	0.23	0.59	0.9	7.33	1.56	7.15	0.04	0.27
0.24	0.24	0.67	1.1	7.85	1.61	7.66	0.04	0.28
0.25	0.25	0.74	1.2	8.37	1.66	8.17	0.04	0.29
0.26	0.26	0.83	1.4	8.89	1.71	8.68	0.05	0.31
0.27	0.27	0.92	1.6	9.41	1.77	9.19	0.05	0.32
0.28	0.28	1.01	1.8	9.93	1.82	9.70	0.05	0.33
0.29	0.29	1.11	2.1	10.45	1.87	10.21	0.05	0.34
0.30	0.30	1.22	2.3	10.97	1.93	10.73	0.06	0.36
0.31	0.31	1.33	2.6	11.48	1.98	11.24	0.06	0.37
0.32	0.32	1.44	2.9	12.00	2.03	11.75	0.06	0.38
0.33	0.33	1.56	3.2	12.52	2.08	12.26	0.07	0.40
0.34	0.34	1.69	3.6	13.04	2.13	12.77	0.07	0.41
0.35	0.35	1.82	4.0	13.56	2.18	13.28	0.07	0.42
0.36	0.36	1.95	4.4	14.08	2.23	13.79	0.08	0.44
0.37	0.37	2.09	4.8	14.60	2.28	14.31	0.08	0.45
0.38	0.38	2.24	5.2	15.12	2.33	14.82	0.08	0.46
0.39	0.39	2.39	5.7	15.64	2.38	15.33	0.09	0.48
0.40	0.40	2.54	6.2	16.16	2.43	15.84	0.09	0.49
0.41	0.41	2.71	6.7	16.68	2.48	16.35	0.10	0.51
0.42	0.42	2.87	7.3	17.20	2.53	16.86	0.10	0.52
0.43	0.43	3.04	7.8	17.72	2.58	17.37	0.10	0.53
0.44	0.44	3.22	8.4	18.23	2.62	17.89	0.11	0.55
0.45	0.45	3.40	9.1	18.75	2.67	18.40	0.11	0.56

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	10.4	19.79	2.76	19.42	0.12	0.59
0.48	0.48	3.98	11.2	20.29	2.81	19.91	0.12	0.60
0.49	0.49	4.18	11.9	20.79	2.86	20.40	0.13	0.62
0.50	0.50	4.38	12.7	21.29	2.91	20.89	0.13	0.63
0.51	0.51	4.59	13.6	21.78	2.95	21.38	0.14	0.65

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0121

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.27	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.43	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.57	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.68	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.79	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.90	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.99	1.09	0.02	0.09
0.08	0.08	0.05	0.1	1.32	1.09	1.25	0.02	0.10
0.09	0.09	0.06	0.1	1.48	1.18	1.41	0.02	0.11
0.10	0.10	0.08	0.1	1.64	1.26	1.56	0.02	0.12
0.11	0.11	0.09	0.1	1.81	1.34	1.72	0.03	0.14
0.12	0.12	0.11	0.2	1.97	1.42	1.88	0.03	0.15
0.13	0.13	0.13	0.2	2.14	1.50	2.03	0.04	0.17
0.14	0.14	0.16	0.2	2.66	1.45	2.54	0.03	0.17
0.15	0.15	0.18	0.3	3.18	1.43	3.06	0.03	0.18
0.16	0.16	0.22	0.3	3.70	1.45	3.57	0.03	0.19
0.17	0.17	0.25	0.4	4.22	1.48	4.08	0.03	0.20
0.18	0.18	0.30	0.5	4.73	1.52	4.59	0.04	0.22
0.19	0.19	0.35	0.5	5.25	1.57	5.10	0.04	0.23
0.20	0.20	0.40	0.6	5.77	1.62	5.61	0.04	0.24
0.21	0.21	0.46	0.8	6.29	1.68	6.12	0.04	0.25
0.22	0.22	0.52	0.9	6.81	1.74	6.64	0.05	0.27
0.23	0.23	0.59	1.1	7.33	1.79	7.15	0.05	0.28
0.24	0.24	0.67	1.2	7.85	1.85	7.66	0.05	0.29
0.25	0.25	0.74	1.4	8.37	1.92	8.17	0.06	0.31
0.26	0.26	0.83	1.6	8.89	1.98	8.68	0.06	0.32
0.27	0.27	0.92	1.9	9.41	2.04	9.19	0.06	0.33
0.28	0.28	1.01	2.1	9.93	2.10	9.70	0.07	0.35
0.29	0.29	1.11	2.4	10.45	2.16	10.21	0.07	0.36
0.30	0.30	1.22	2.7	10.97	2.22	10.73	0.08	0.38
0.31	0.31	1.33	3.0	11.48	2.28	11.24	0.08	0.39
0.32	0.32	1.44	3.4	12.00	2.34	11.75	0.09	0.41
0.33	0.33	1.56	3.7	12.52	2.40	12.26	0.09	0.42
0.34	0.34	1.69	4.1	13.04	2.46	12.77	0.09	0.43
0.35	0.35	1.82	4.6	13.56	2.52	13.28	0.10	0.45
0.36	0.36	1.95	5.0	14.08	2.58	13.79	0.10	0.46
0.37	0.37	2.09	5.5	14.60	2.63	14.31	0.11	0.48
0.38	0.38	2.24	6.0	15.12	2.69	14.82	0.11	0.49
0.39	0.39	2.39	6.6	15.64	2.75	15.33	0.12	0.51
0.40	0.40	2.54	7.1	16.16	2.80	15.84	0.12	0.52
0.41	0.41	2.71	7.7	16.68	2.86	16.35	0.13	0.54
0.42	0.42	2.87	8.4	17.20	2.92	16.86	0.13	0.55
0.43	0.43	3.04	9.0	17.72	2.97	17.37	0.14	0.57
0.44	0.44	3.22	9.7	18.23	3.03	17.89	0.14	0.58
0.45	0.45	3.40	10.5	18.75	3.08	18.40	0.15	0.60

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	12.0	19.79	3.19	19.42	0.16	0.63
0.48	0.48	3.98	12.9	20.29	3.24	19.91	0.16	0.64
0.49	0.49	4.18	13.8	20.79	3.30	20.40	0.17	0.66
0.50	0.50	4.38	14.7	21.29	3.35	20.89	0.17	0.67
0.51	0.51	4.59	15.7	21.78	3.41	21.38	0.18	0.69

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0157

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.31	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.49	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.64	0.47	0.01	0.04
0.04	0.04	0.01	0.0	0.66	0.78	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.91	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	1.02	0.94	0.02	0.08
0.07	0.07	0.04	0.0	1.15	1.13	1.09	0.02	0.09
0.08	0.08	0.05	0.1	1.32	1.24	1.25	0.02	0.10
0.09	0.09	0.06	0.1	1.48	1.34	1.41	0.03	0.12
0.10	0.10	0.08	0.1	1.64	1.44	1.56	0.03	0.13
0.11	0.11	0.09	0.1	1.81	1.53	1.72	0.04	0.15
0.12	0.12	0.11	0.2	1.97	1.62	1.88	0.04	0.16
0.13	0.13	0.13	0.2	2.14	1.71	2.03	0.05	0.18
0.14	0.14	0.16	0.3	2.66	1.65	2.54	0.04	0.18
0.15	0.15	0.18	0.3	3.18	1.63	3.06	0.04	0.19
0.16	0.16	0.22	0.4	3.70	1.65	3.57	0.04	0.20
0.17	0.17	0.25	0.4	4.22	1.69	4.08	0.04	0.21
0.18	0.18	0.30	0.5	4.73	1.73	4.59	0.05	0.23
0.19	0.19	0.35	0.6	5.25	1.79	5.10	0.05	0.24
0.20	0.20	0.40	0.7	5.77	1.85	5.61	0.05	0.25
0.21	0.21	0.46	0.9	6.29	1.91	6.12	0.06	0.27
0.22	0.22	0.52	1.0	6.81	1.98	6.64	0.06	0.28
0.23	0.23	0.59	1.2	7.33	2.04	7.15	0.06	0.29
0.24	0.24	0.67	1.4	7.85	2.11	7.66	0.07	0.31
0.25	0.25	0.74	1.6	8.37	2.18	8.17	0.07	0.32
0.26	0.26	0.83	1.9	8.89	2.25	8.68	0.08	0.34
0.27	0.27	0.92	2.1	9.41	2.32	9.19	0.08	0.35
0.28	0.28	1.01	2.4	9.93	2.39	9.70	0.09	0.37
0.29	0.29	1.11	2.7	10.45	2.46	10.21	0.09	0.38
0.30	0.30	1.22	3.1	10.97	2.53	10.73	0.10	0.40
0.31	0.31	1.33	3.4	11.48	2.60	11.24	0.10	0.41
0.32	0.32	1.44	3.8	12.00	2.67	11.75	0.11	0.43
0.33	0.33	1.56	4.3	12.52	2.73	12.26	0.12	0.45
0.34	0.34	1.69	4.7	13.04	2.80	12.77	0.12	0.46
0.35	0.35	1.82	5.2	13.56	2.87	13.28	0.13	0.48
0.36	0.36	1.95	5.7	14.08	2.93	13.79	0.13	0.49
0.37	0.37	2.09	6.3	14.60	3.00	14.31	0.14	0.51
0.38	0.38	2.24	6.9	15.12	3.07	14.82	0.15	0.53
0.39	0.39	2.39	7.5	15.64	3.13	15.33	0.15	0.54
0.40	0.40	2.54	8.1	16.16	3.19	15.84	0.16	0.56
0.41	0.41	2.71	8.8	16.68	3.26	16.35	0.16	0.57
0.42	0.42	2.87	9.5	17.20	3.32	16.86	0.17	0.59
0.43	0.43	3.04	10.3	17.72	3.38	17.37	0.18	0.61
0.44	0.44	3.22	11.1	18.23	3.45	17.89	0.18	0.62
0.45	0.45	3.40	11.9	18.75	3.51	18.40	0.19	0.64

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	13.7	19.79	3.63	19.42	0.20	0.67
0.48	0.48	3.98	14.7	20.29	3.69	19.91	0.21	0.69
0.49	0.49	4.18	15.7	20.79	3.76	20.40	0.22	0.71
0.50	0.50	4.38	16.7	21.29	3.82	20.89	0.23	0.73
0.51	0.51	4.59	17.8	21.78	3.88	21.38	0.23	0.74

Single A inlet, in sump condition with curb openings on both sides:
Open Area (for orifice calc in sq. ft.): 3.9314236
Length of Weir (feet): 11.354167

Head (ft)	Head (in)	Weir Q	Orifice Q	Control Q
0.05	0.6	0.34	4.23	0.34
0.1	1.2	0.96	5.99	0.96
0.15	1.8	1.77	7.33	1.77
0.2	2.4	2.72	8.47	2.72
0.25	3	3.80	9.46	3.80
0.3	3.6	5.00	10.37	5.00
0.35	4.2	6.30	11.20	6.30
0.4	4.8	7.70	11.97	7.70
0.45	5.4	9.19	12.70	9.19
0.5	6	10.76	13.39	10.76
0.55	6.6	12.41	14.04	12.41
0.6	7.2	14.14	14.66	14.14
0.65	7.8	15.95	15.26	15.26
0.67	8.04	16.69	15.49	15.49
0.7	8.4	17.82	15.84	15.84
0.75	9	19.76	16.39	16.39
0.8	9.6	21.77	16.93	16.93
0.85	10.2	23.85	17.45	17.45
0.9	10.8	25.98	17.96	17.96
0.95	11.4	28.18	18.45	18.45
1	12	30.43	18.93	18.93
1.05	12.6	32.74	19.40	19.40
1.1	13.2	35.11	19.85	19.85
1.15	13.8	37.53	20.30	20.30
1.2	14.4	40.00	20.74	20.74

Calculation of open area:

	(in^2)	(ft^2)
Total Grate Area	1000	6.9444444
Cross Bar Area	-366	-2.541667
Supports (ends)	-115.625	-0.802951
Areas Counted Twice	47.75	0.3315972
	566.125	3.9314236

Calculation of Length of Weir:

	(in)	(ft)
Total Perimeter of Grate	90	7.5
Short Cross Bars	-3.5	-0.291667
End Supports	-9.25	-0.770833
Bearing Bars	-13	-1.083333
Curb Openings	72	6
	136.25	11.354167

Single D inlet, in sump condition:

Open Area (for orifice calc in sq. ft.): 3.93

Length of Weir (feet): 8.40

Head (ft)	Head (in)	Weir Q	Orifice Q	Control Q
0.05	0.6	0.25	4.23	0.25
0.1	1.2	0.71	5.99	0.71
0.15	1.8	1.31	7.33	1.31
0.2	2.4	2.01	8.47	2.01
0.25	3	2.81	9.46	2.81
0.3	3.6	3.70	10.37	3.70
0.35	4.2	4.66	11.20	4.66
0.4	4.8	5.69	11.97	5.69
0.45	5.4	6.79	12.70	6.79
0.5	6	7.96	13.39	7.96
0.55	6.6	9.18	14.04	9.18
0.6	7.2	10.46	14.66	10.46
0.65	7.8	11.79	15.26	11.79
0.67	8.04	12.34	15.49	12.34
0.7	8.4	13.18	15.84	13.18
0.75	9	14.61	16.39	14.61
0.8	9.6	16.10	16.93	16.10
0.85	10.2	17.63	17.45	17.45
0.9	10.8	19.21	17.96	17.96
0.95	11.4	20.83	18.45	18.45
1	12	22.50	18.93	18.93
1.05	12.6	24.21	19.40	19.40
1.1	13.2	25.96	19.85	19.85
1.15	13.8	27.75	20.30	20.30
1.2	14.4	29.58	20.74	20.74
1.25	15	31.45	21.16	21.16
1.3	15.6	33.35	21.58	21.58

Calculation of open area:

	(in^2)	(ft^2)
Total Grate Area	1000	6.944
Cross Bar Area	-366	-2.542
Supports (ends)	-115.625	-0.803
Areas Counted Twice	47.75	0.332
	566.125	3.931

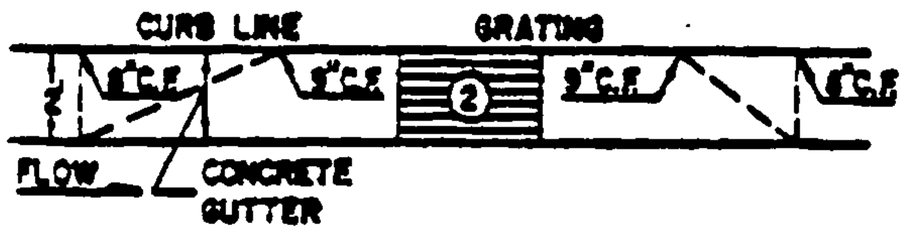
Calculation of Length of Weir (four sides open):

	(in)	(ft)
Total Perimeter of Grate	130	10.833
Short Cross Bars	-7	-0.583
End Supports	-9.25	-0.771
Bearing Bars	-13	-1.083
	100.75	8.396
		0.775

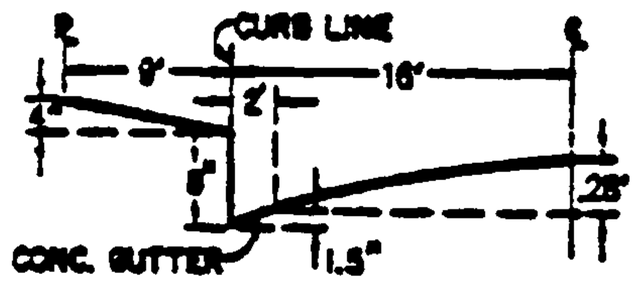
Calculation of Length of Weir (three sides open):

	(in)	(ft)
Total Perimeter of Grate	90	7.500
Short Cross Bars	-3.5	-0.292
End Supports	-9.25	-0.771
Bearing Bars	-13	-1.083
	64.25	5.354
		0.714

GRATING CAPACITIES FOR TYPE 'A', 'C' and 'D'



GRATING & GUTTER PLAN



TYPICAL HALF STREET SECTION
(ABOVE BASIN)

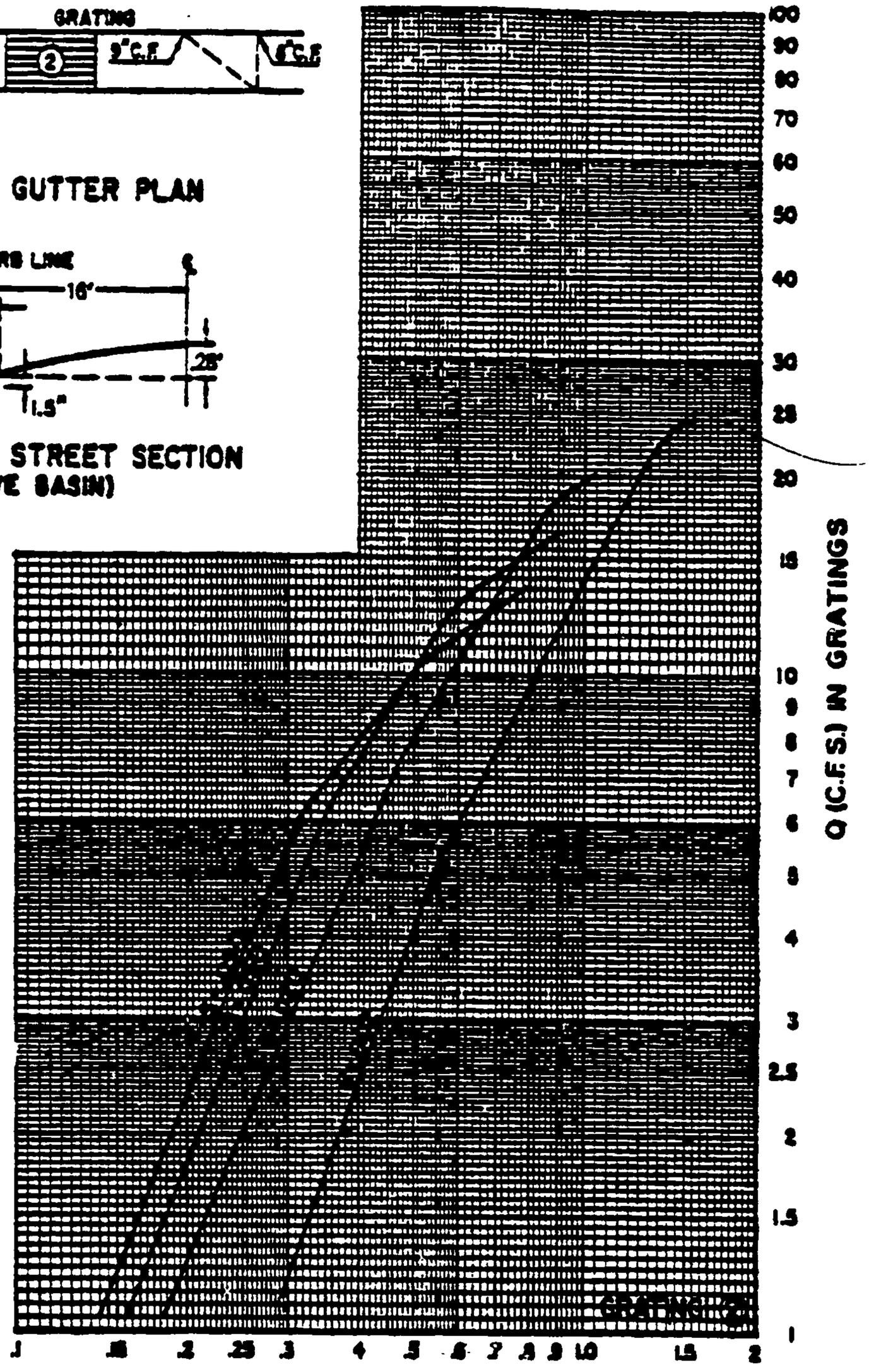


PLATE 22.3 D-5

APPENDIX D

Hydraulic Grade Line Profiles

POT HOLE DATA					
PH#	UTIL	NORTHING	EASTING	ELEV	DEPTH
17	GAS	1475906 51	1556309 45	5468 69	32"
26	WATER	1475688 75	1556385 16	5467 98	67"

⚠ UTILITY CROSSING TABLE

CROSSING	SD	GAS	SAS	WL	CLEARANCE
1	TP=64 55'	BP=65 87'	N/A	N/A	1 32'
2	BP=59 57'	N/A	TP=57 65'	N/A	1.92'
3	TP=61 41'	N/A	N/A	BP=62.36'	0.95'
4	TP=66 03'	BP=67 01'	N/A	N/A	0.98'
5	BP=64 84'	N/A	TP=62.14'	N/A	2 70'
6 *	BP=64 89'	N/A	N/A	TP=64 48'	0 41'
7	TP=67 11'	BP=67 88'	N/A	N/A	0 77

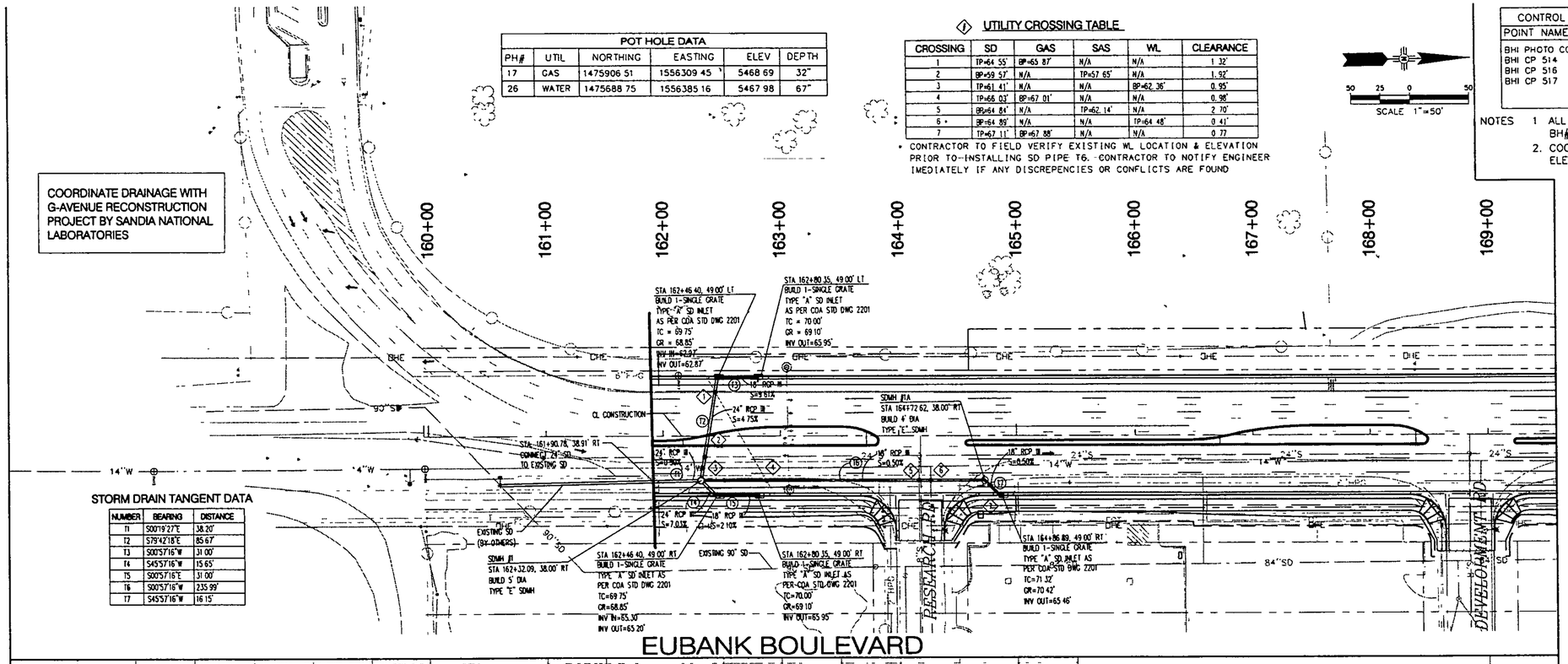
* CONTRACTOR TO FIELD VERIFY EXISTING WL LOCATION & ELEVATION PRIOR TO-INSTALLING SD PIPE T6. -CONTRACTOR TO NOTIFY ENGINEER IMEDIATELY IF ANY DISCREPENCIES OR CONFLICTS ARE FOUND

CONTROL POINT DATA TABLE (GROUND COORDINATES)			
POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956 90	1555960 12	5463 22
BHI CP 516	1478043 06	1555862 87	5461.77
BHI CP 517	1477917 89	1557166 71	5480 08

NOTES
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM

50 25 0 50
SCALE 1"=50'

COORDINATE DRAINAGE WITH G-AVENUE RECONSTRUCTION PROJECT BY SANDIA NATIONAL LABORATORIES

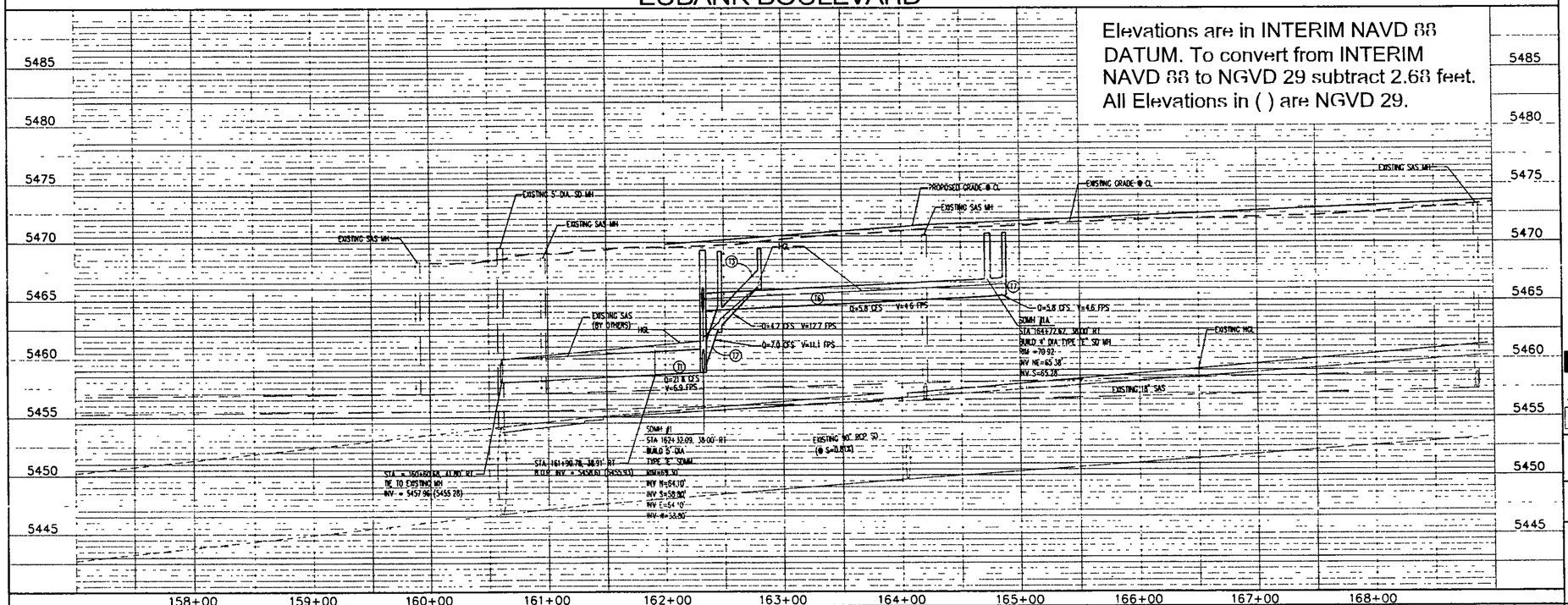


STORM DRAIN TANGENT DATA

NUMBER	BEARING	DISTANCE
T1	S00°19'27"E	38.20'
T2	S79°42'18"E	85 67'
T3	S00°57'16"W	31 00'
T4	S45°57'16"W	15 65'
T5	S00°57'16"E	31 00'
T6	S00°57'16"W	235 99'
T7	S45°57'16"W	16 15'

EUBANK BOULEVARD

Elevations are in INTERIM NAVD 88 DATUM. To convert from INTERIM NAVD 88 to NGVD 29 subtract 2.68 feet. All Elevations in ( ) are NGVD 29.



SCALE 1" = 50' (HORIZ)
1" = 5' (VERT)

P:\00258\Hydro\Design\PNP\00258pnp01 dwg
Jun 19, 2002 - 4 22pm

BHI JOB NO 00258

MARCH 2002

BENCH MARKS

ACS Aluminum Cap stamped "6-L21, 1986/92"
Intersection of Central Ave & Elizabeth St. Geographic Position (NAD 1927), in feet N M State Plane Coordinates (Central Zone) X=417,975 01, Y=1,481,220 09 (Grid) Elevation=5500 67 NGVD29, in feet Interim NAD 83 in feet [X= 1,558,774 59, Y= 1,481,809 99] (Ground)
Ground-to-grid factor 0 99964527
Transformed to Interim NAVD 88 by Carpscon V5 11 08
[Elevation= 5503 35 Interim NAVD88, in feet]

AS-BUILT INFORMATION

CONTRACTOR — DATE
WORK STAKED BY — DATE
INSPECTOR'S ACCEPTANCE BY — DATE
FIELD VERIFICATION BY — DATE
DRAWINGS CORRECTED BY — DATE
MICRO-FILM INFORMATION
RECORDED BY — DATE
NO

SURVEY INFORMATION

FIELD NOTES

NO.	BY	DATE

ENGINEER'S SEAL

No	Date	REMARKS	By
		REVISIONS	
		DESIGN	

Designed By MPC DATE 04/02
Drawn By SFC DATE 04/02
Checked By MPC/CWH DATE 04/02

Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

EUBANK BOULEVARD IMPROVEMENTS
STORM DRAIN
STA. 160+00 TO STA. 169+00

Design Review Committee	City Engineer Approval	Last Design Update	Mo/Day/Yr	Mo/Day/Yr

City Project No	Zone Map No	Sheet	Of
6061.01	L-21&M-21	35	114

UTILITY CROSSING TABLE

CROSSING	SD	GAS	SAS	WL	COMM	CLEARANCE
1	BP=69 83'	N/A	TP=67 26'	N/A	TP=69 43'	2 57' / 0 40'
2	BP=69 85'	N/A	N/A	TP=68 78'	N/A	1 07'
3	TP=71 70'	N/A	N/A	N/A	BP=72 13'	0. 43'
4	BP=62 24'	N/A	TP=62 04'	N/A	N/A	0. 20'
5	TP=64 73'	N/A	N/A	N/A	BP=68 07' / BP=67 82'	3 34' / 3 09'
6	TP=64 77'	N/A	N/A	N/A	BP=69 69'	4 92'

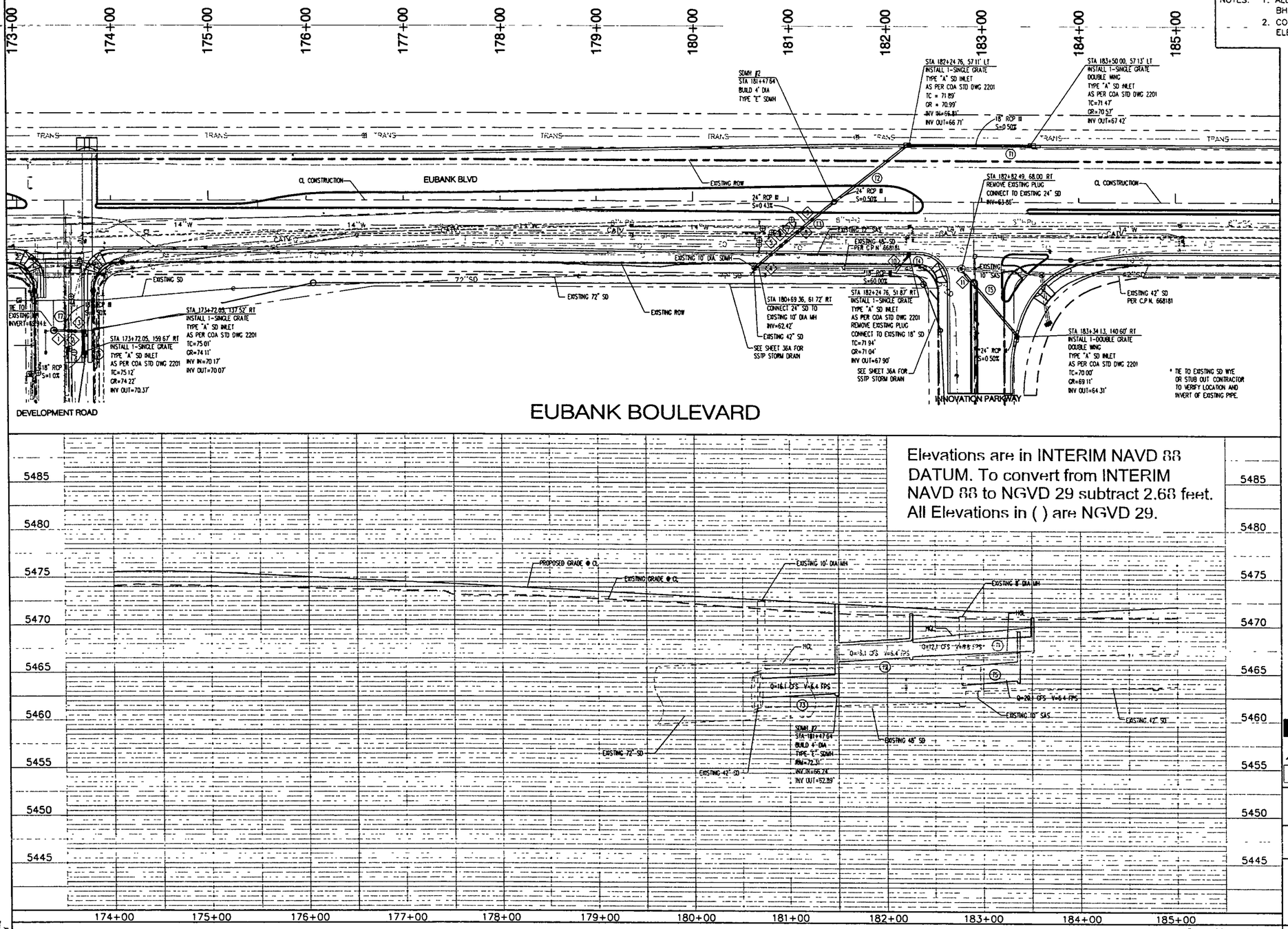
UTILITY CROSSING TABLE

CROSSING	SD	GAS	SAS	WL	COMM	CLEARANCE
7			CONFLICT WITH CATV, MUST ADJUST CATV			
8	TP=64 88'	N/A	N/A	BP=65 09'	N/A	0 21'
9	TP=64 95'	BP=67 51'	N/A	N/A	N/A	2 56'
10	BP=67 73'	N/A	TP=62 53'	N/A	N/A	5 20'
11	BP=63 75'	N/A	TP=63. 25'	N/A	N/A	0 50'

CONTROL POINT DATA TABLE (GROUND COORDINATES)

POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956 90	1555960 12	5463 22
BHI CP 516	1478043 06	1555862 87	5461 77
BHI CP 517	1477917 89	1557166 71	5480 08

NOTES.
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BHI# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM



STORM DRAIN TANGENT DATA

NUMBER	BEARING	DISTANCE
T1	S00°56'59"W	122.32'
T2	S37°15'43"E	94 23'
T3	S39°04'49"E	108 28'
T4	S44°02'46"E	8.00'
T5	S54°02 58"W	89 53
T6	N88°55'26"W	19 33'
T7	S03°09'53"W	25.76'

POT HOLE DATA

PH#	UTIL	NORTHING	EASTING	ELEV	DEPTH
5	GAS	1477795.48	1556412.00	5471 29	34"
6	CATV	1477774 30	1556424 92	5471 60	81"
7	WATER	1477764 59	1556420 66	5471.59	58"
8	F O	1477760 41	1556437 55	5472.29	28"
9	COMM	1477761 00	1556432 81	5472 24	24"

POT HOLE DATA

PH#	UTIL	NORTHING	EASTING	ELEV	DEPTH
10	GAS	1477014 20	1556550.24	5475 73	36"
11	ELEC	1477007 06	1556561 67	5475.91	38"
12	F O	1477004 45	1556561 57	5475.92	18"
13	WATER	1477048.26	1556558 38	5474 96	CLEAR TO 96"
27	WATER	1477049 16	1556481 43	5474 67	72"
28	COMM	1477051 18	1556540 07	5475 33	33"

AS-BUILT INFORMATION — CONTRACTOR / STAKED BY / FIELD VERIFICATION BY / DRAWINGS CORRECTED BY — DATE

BENCH MARKS: ACS Aluminum Cap stamped "6-121, 1986/92"

SURVEY INFORMATION: Intersection of Central Ave & Elizabeth St Geographic Position (NAD 1927), in feet N M State Plane Coordinates (Central Zone) X=417,975 01, Y=1,481,220 09 (Grid) Elevation=5500 67 NGVD29, in feet Interim NAD 83, in feet |X= 1,558,774 59, Y= 1,481,809 991 (Ground) Ground-to-grid factor 0 99964527 Transformed to Interim NAVD 88 by Corpscon V5 11 08 [Elevation= 5503 35 Interim NAVD88, in feet]

MICRO-FILM INFORMATION: RECORDED BY — DATE; NO.

FIELD NOTES: NO. | BY | DATE

ENGINEER'S SEAL

No.	Date	REMARKS	By
		REVISIONS	

DESIGN: Designed By M.P.C DATE 04/02; Drawn By MTD DATE 04/02; Checked By M.P.C./C.W.H. DATE 04/02

Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

EUBANK BOULEVARD IMPROVEMENTS
STORM DRAIN
STA. 174+00 TO STA. 185+00

Design Review Committee	City Engineer Approval	Last Design Update	Mo./Day/Yr	Mo./Day/Yr

City Project No	Zone Map No	Sheet	Of
6061.01	L-21&M-21	36	114

BHI JOB NO 00268

MARCH 2002

DWG NAME: P:\00268\Hydro\Design\PNP\00268pnp02.dwg
Jan 13, 2002 - 4 23pm

APPENDIX F

Correspondence

Bohannan ▲ Huston

TELEPHONE CONFERENCE NOTES

JOB NO. 00268

PROJECT TITLE Eubank

DATE **9/24/01**

TIME 11:40

CONVERSATION WITH Brad Bingham **OF** City of Albuquerque

AND John Gurule **OF** BHI

SUBJECT **Size of trunk line in area north of Southern Ave.**

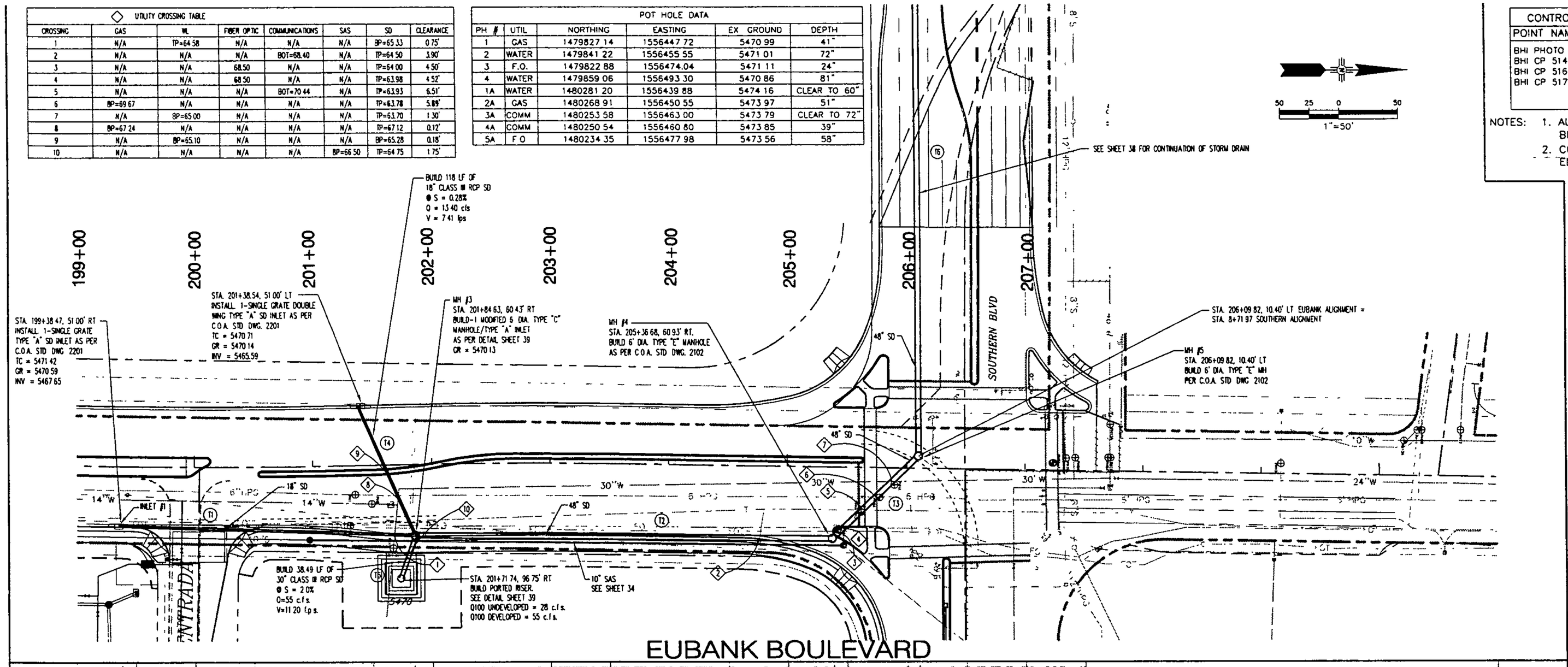
CONTENT

I asked Brad if it was acceptable to use an 18 inch diameter pipe for the trunk line in the area north of Southern Avenue instead of the standard 24 inch pipe. His response was, "since the trunk line would be connecting to an existing 18-inch pipe in the Acoma area, it would not be beneficial to the City, to place a 24-inch pipe upstream and then narrow it down to 18 inches at the connection."

John Gurule

UTILITY CROSSING TABLE							
CROSSING	GAS	WL	FIBER OPTIC	COMMUNICATIONS	SAS	SD	CLEARANCE
1	N/A	TP=64.58	N/A	N/A	N/A	BP=65.33	0.75'
2	N/A	N/A	N/A	BOT=68.40	N/A	TP=64.50	3.90'
3	N/A	N/A	68.50	N/A	N/A	TP=64.00	4.50'
4	N/A	N/A	68.50	N/A	N/A	TP=63.98	4.52'
5	N/A	N/A	N/A	BOT=70.44	N/A	TP=63.93	6.51'
6	BP=69.67	N/A	N/A	N/A	N/A	TP=63.78	5.89'
7	N/A	BP=65.00	N/A	N/A	N/A	TP=63.70	1.30'
8	BP=67.24	N/A	N/A	N/A	N/A	TP=67.12	0.12'
9	N/A	BP=65.10	N/A	N/A	N/A	BP=65.28	0.18'
10	N/A	N/A	N/A	N/A	BP=66.50	TP=64.75	1.75'

POT HOLE DATA					
PH #	UTIL	NORTHING	EASTING	EX GROUND	DEPTH
1	GAS	1479827.14	1556447.72	5470.99	41"
2	WATER	1479841.22	1556455.55	5471.01	72"
3	F.O.	1479822.88	1556474.04	5471.11	24"
4	WATER	1479859.06	1556493.30	5470.86	81"
1A	WATER	1480281.20	1556439.88	5474.16	CLEAR TO 60"
2A	GAS	1480268.91	1556450.55	5473.97	51"
3A	COMM	1480253.58	1556463.00	5473.79	CLEAR TO 72"
4A	COMM	1480250.54	1556460.80	5473.85	39"
5A	F.O.	1480234.35	1556477.98	5473.56	58"

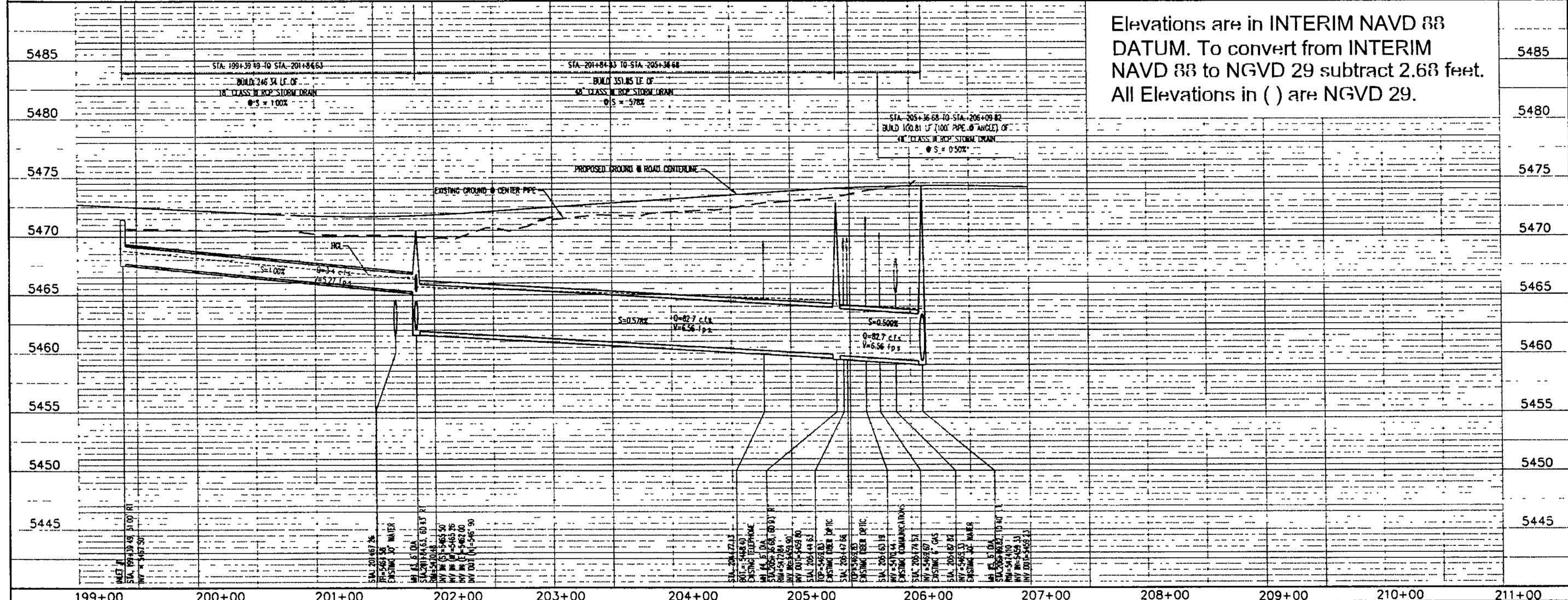


CONTROL POINT DATA TABLE (GROUND COORDINATES)			
POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956.90	1555960.12	5463.22
BHI CP 516	1478043.06	1555862.87	5461.77
BHI CP 517	1477917.89	1557166.71	5480.08

NOTES:
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM

STORM DRAIN TANGENT DATA

NUMBER	BEARING	DISTANCE
T1	N01°51'44"E	246.26'
T2	N00°31'33"E	351.85'
T3	N44°07'12"W	100.81'
T4	S66°38'37"W	118.07'
T5	N71°53'15"W	38.49'



Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

EUBANK BOULEVARD IMPROVEMENTS
STORM DRAIN PLAN & PROFILE - LA ENTRADA TO SOUTHERN
STA. 199+00 TO STA. 211+00

Design Review Committee	City Engineer Approval	Last Design Update	mo./Day/Yr	mo./Day/Yr

City Project No	Zone Map No.	Sheet	Of
6061.01	L-21&M-21	37	114

BHI JOB NO 00268

MARCH 2002

AS-BUILT INFORMATION

	DATE
CONTRACTOR	
WORK STAKED BY	DATE
INSPECTOR'S ACCEPTANCE BY	DATE
FIELD VERIFICATION BY	DATE
DRAWINGS CORRECTED BY	DATE

MICRO-FILM INFORMATION

	DATE
RECORDED BY	
NO	

BENCH MARKS
ACS Aluminum Cap stamped "B-L21, 1986/92"
Intersection of Central Ave. & Elizabeth St. Geographic Position (NAD 1927), in feet N.M. State Plane Coordinates (Central Zone) X=417,975.01, Y=1,481,220.09 (Grid) Elevation=5500.67 NGVD29, in feet Interim NAD 83, in feet [X= 1,558,774.59, Y= 1,481,809.99] (Ground)
Ground-to-grid factor 0.99964527
Transformed to Interim NAVD 88 by Carpscon V5.11.08
[Elevation= 5503.35 Interim NAVD88, in feet]

SURVEY INFORMATION
FIELD NOTES

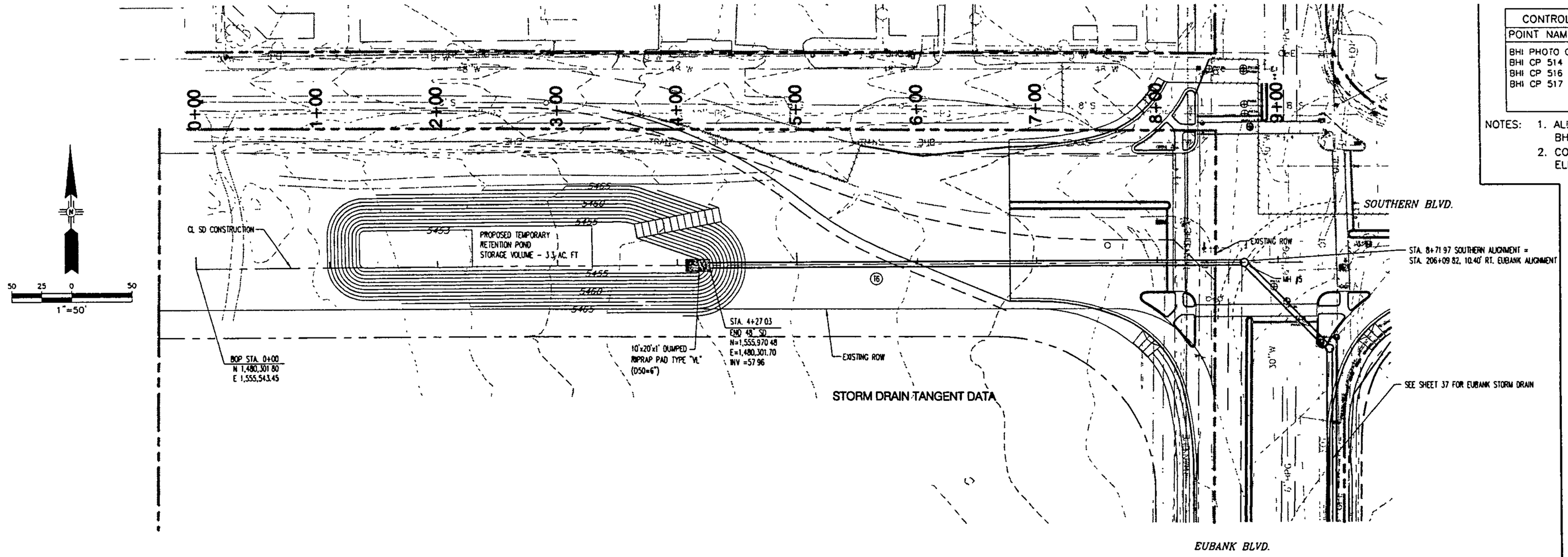
NO	BY	DATE

ENGINEER'S SEAL

No	Date	REMARKS	By
		REVISIONS	
		DESIGN	

Designed By M.P.C. DATE 04/02
Drawn By MTD DATE 04/02
Checked By M.P.C./C.W.H. DATE 04/02

DWG_NAME MAGDYY
P:\00268\Hydro\Design\PNP\00268pnp03-rev.dwg
Jun 19, 2002 - 4:23pm



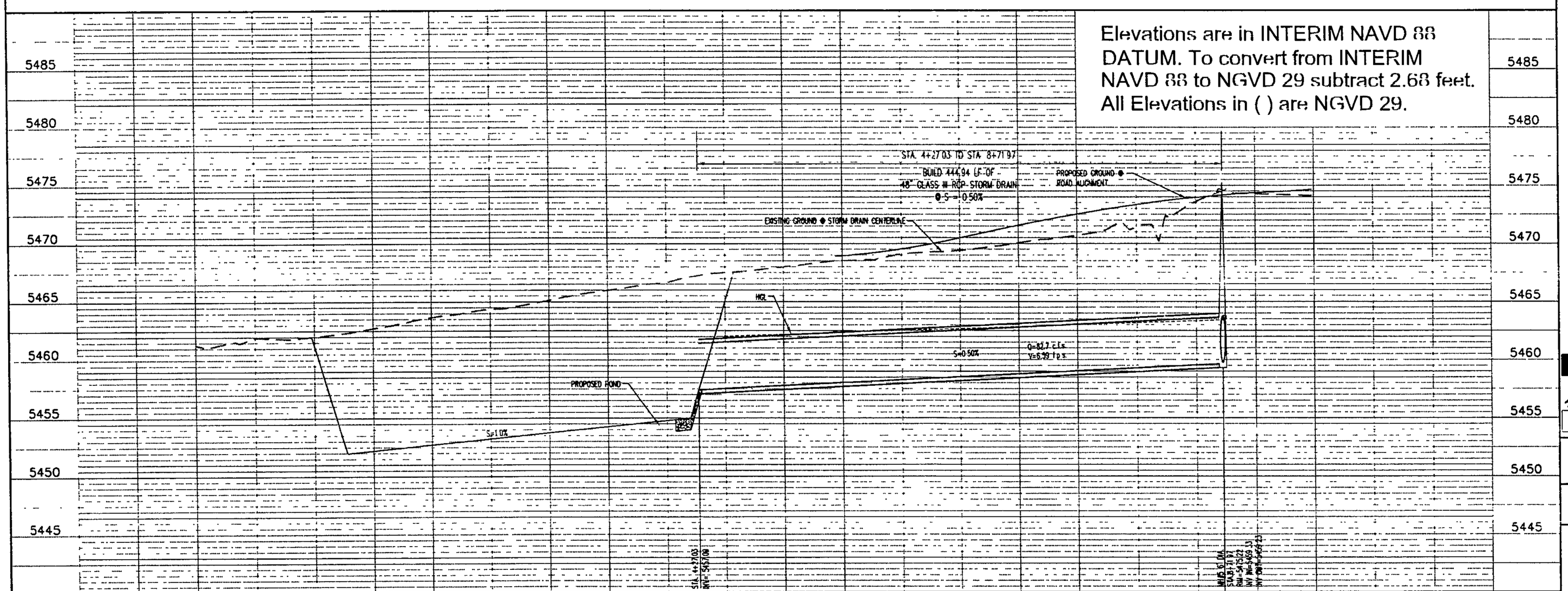
CONTROL POINT DATA TABLE (GROUND COORDINATES)			
POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956.90	1555960.12	5463.22
BHI CP 516	1478043.06	1555862.87	5461.77
BHI CP 517	1477917.89	1557166.71	5480.08

NOTES:
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM

STORM DRAIN PIPE TANGENT DATA

NUMBER	BEARING	DISTANCE
TG	N89°59'09"W	444.94'

SOUTHERN BOULEVARD



AS-BUILT INFORMATION

	DATE
CONTRACTOR	
WORK STAKED BY	DATE
INSPECTOR'S ACCEPTANCE BY	DATE
FIELD COMPLETION BY	DATE
DRAWINGS CORRECTED BY	DATE
MICRO-FILM INFORMATION	
RECORDED BY	DATE
NO	

BENCH MARKS

ACS Aluminum Cap stamped "B-L21, 1986/92"
Intersection of Central Ave. & Elizabeth St. Geographic Position (NAD 1927), in feet N.M. State Plane Coordinates (Central Zone) X=417,975.01, Y=1,481,220.09 (Grid) Elevation=5500.67 NGVD29, in feet Interim NAD 83, in feet [X= 1,558,774.59, Y= 1,481,809.99] (Ground) Ground-to-grid factor 0.99964527 Transformed to Interim NAVD 88 by Carpscan V5.11.08 [Elevation= 5503.35 Interim NAVD88, in feet]

SURVEY INFORMATION

FIELD NOTES

NO.	BY	DATE

ENGINEER'S SEAL

No.	Date	REMARKS	By
		REVISIONS	

DESIGN

Designed By M.P.C. DATE 04/02
Drawn By MTD DATE 04/02
Checked By M.P.C./C.W.H. DATE 04/02

Bohannan ▲ Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

EUBANK BOULEVARD IMPROVEMENTS
DETENTION POND OUTFALL PLAN & PROFILE
STA. 0+00 TO STA. 9+00

Design Review Committee	City Engineer Approval	Last Design Update	Mo./Day/Yr	Mo./Day/Yr

City Project No	Zone Map No	Sheet	Of
6061.01	L-21&M-21	38	114

BHI JOB NO 00269

MARCH 2002

DWG. NAME MM/D/YY
P:\00269\Hydro\Design\PNP\00269pns04-rev.dwg
Jun 19, 2002 - 4:24pm

APPENDIX E

Eubank Boulevard. 54-inch Storm Drain As-Builts

BCHA [illegible]

Courtyard One

7500 [illegible]

Albuquerque

NEW [illegible]

voice 505 823 [illegible]

[illegible]

www.bhinc.com

EUBANK BOULEVARD IMPROVEMENTS, KIRTLAND AIR FORCE BASE TO CENTRAL AVENUE DRAINAGE REPORT

July 9, 2002

PREPARED FOR:

CITY OF ALBUQUERQUE
P.O. BOX 1293
ALBUQUERQUE, NM 87103

L-21/D054

Bohannan △ Huston INC.

EUBANK BOULEVARD IMPROVEMENTS,
KIRTLAND AIR FORCU BASE TO CENTRAL AVENUE
DRAINAGE REPORT

I, Craig W. Hoover, hereby certify that I am a Register Professional Engineer, registered in the state of New Mexico, and that the following report was prepared under my direction and is true and correct to the best of my knowledge and belief.

Craig W. Hoover 7/9/02

Craig W. Hoover, P.E.

NMPE No. 11848

CRAIG W. HOOVER
NEW MEXICO
11848
REGISTERED PROFESSIONAL ENGINEER

TABLE OF CONTENTS

I. INTRODUCTION 1
II. METHODOLOGIES AND REFERENCES 1
III. EXISTING HYDRAULIC/HYDROLOGIC CONDITIONS 1
IV. STORM DRAIN DESIGN 2
A. Hydrologic Analysis and Results 2
B. Proposed Drainage Improvements 3
C. Hydraulic Analysis of Proposed Systems 8
V. CONCLUSION 10

TABLES

TABLE 1 3
TABLE 2 4

APPENDICES

APPENDIX A - Excerpts from the "City of Albuquerque Development Process Manual"

APPENDIX B - Plates

PLATE 1 – Drainage Basin Map

PLATE 2 – Drainage Basin Map

APPENDIX C - Calculations

Drainage Basin Hydrology Calculations

Typical Street Section

Street Section Capacity Calculations

Sump Inlet (Type A and Type D) Capacity Calculations

On-Grade Inlet Capacity Nomograph

APPENDIX D - Hydraulic Grade Line Profiles

APPENDIX E - Eubank Boulevard. 54-inch Storm Drain As-Builts

APPENDIX F - Correspondence

I. INTRODUCTION

The purpose of this report is to address the drainage improvements for Eubank Boulevard from Kirtland Air Force Base to Central Avenue. Included in this report:

- data supporting the proposed Eubank Boulevard. storm drain improvements.
- data supporting the proposed size and location of the temporary retention pond.

The basis for this report is the approved "Manzano Mesa Master Drainage Study", Smith Engineering Company, August 1996.

II. METHODOLOGIES AND REFERENCES

This report is prepared in Accordance with City of Albuquerque Development Process Manual (DPM) criteria; specifically, the procedures outlined in DPM Section 22.2. This report also references the approved "Manzano Mesa Master Drainage Study," Smith Engineering Company, August 1996.

III. EXISTING HYDRAULIC/HYDROLOGIC CONDITIONS

The existing hydraulic conditions along the Eubank Boulevard Improvement Project vary greatly from the northern section of the project to the southern. The property adjacent to Eubank Boulevard from Southern Boulevard to Central Avenue is currently developed and serviced by an existing storm drain system that conveys flows north toward Central Avenue. Curb and gutter exists along this portion of Eubank Boulevard and conveys runoff to catch basins connected to the storm drain.

Eubank Boulevard from Southern Boulevard south to Kirtland AFB is mainly undeveloped with scattered areas of development existing along the east side of the roadway. There is currently no curb and gutter along this section of roadway except for approximately 1400 feet of curb and gutter located from Opportunity Avenue southward. A storm drain parallels Eubank Boulevard from the future Innovation Parkway (Stephen Moody Street) and continues south, past Research Road. This storm drain, designated as System "A" in the Manzano Mesa Master Drainage Plan, currently

receives flow from developments in the area and will also ultimately drain a portion of Central Avenue east of Elizabeth Street.

A FEMA Floodplain of approximately 2.2 acres exists on two parcels of land adjacent to Eubank Boulevard. The land containing the northern portion of the floodplain is owned by the Albuquerque Public Schools (APS) and the southern section of the floodplain is within land belonging to GSL Properties of Portland Oregon. In the near future, the southern parcel of land will be the site of the Manzano Mesa Apartments. The proposed drainage improvements for the apartment complex will remove the floodplain located on the southern parcel of land. The storm drain improvements proposed in this drainage report will remove a majority of the floodplain within the APS property.

IV. STORM DRAIN DESIGN

A. Hydrologic Analysis and Results

The hydrology for the fully developed conditions model was based on the City of Albuquerque Development Process Manual, Section 22.2, Hydrology (DPM). The 10-year storm was the basis for analyzing the street hydraulics and the proposed storm drain was sized to accommodate the 100-year storm. The procedure described in Part A of the DPM was used for the analysis, and excerpts of this procedure are located in Appendix A. This procedure is an accepted method for calculating runoff volumes and peak flow rates from drainage basins that are 40 acres or smaller.

The procedure is based on the Rational Method and initial abstraction/uniform infiltration precipitation losses and utilizes four different land treatments. The land treatments range from undisturbed land (Type A) to impervious surfaces (Type D). Table 1 summarizes the land treatment types utilized for each contributing drainage basin in the fully developed conditions model. Each of the street drainage basins, with the exception of the APS property and basin 1, contains minimal areas considered to be Land Treatment Type C (median

landscaping), but this area was determined to be negligible, and the street basins were therefore modeled as being 100% Land Treatment Type D.

TABLE 1

LAND TREATMENT TYPES AND BASIN FLOWS - FULLY DEVELOPED CONDITIONS

DRAINAGE BASIN	TOTAL AREA (ACRES)	LAND TREATMENT TYPES (%)				PEAK FLOW (CFS)	RUNOFF VOLUME (CF)*
		A	B	C	D		
1	2.53	0	0	10	90	17.7	
2	1.82	0	0	0	100	9.1	
3	3.15	0	0	0	100	14.8	
4	3.21	0	0	0	100	16.1	
5	2.84	0	0	0	100	14.3	48040
6	2.66	0	0	0	100	13.4	45000
7	0.61	0	0	0	100	3.1	
8	0.43	0	0	0	100	2.2	
9	1.38	0	0	0	100	6.9	
10	2.3	0	0	0	100	11.5	
11	1.18	0	0	0	100	6.0	
APS Property	13.30	0	25	25	50	55.0	
APS Property**	13.30	85	0	15	0	28.0	36450

* 100-yr 10 day volume
** Existing conditions assumed for this basin only.

This method also takes into account the Precipitation Zone that the project lies within. The entire Eubank Boulevard project lies within Precipitation Zone 3 as defined by the DPM.

Plates 1 and 2 in Appendix C contain the fully developed conditions basin maps.

B. Proposed Drainage Improvements

The proposed Eubank Boulevard storm drain improvements that will be constructed in two phases were divided into 4 systems based on the locations of high points along Eubank

Boulevard. Table 2 summarizes the basins drained by each of the systems and corresponding phases. Plates 1 and 2 display the limits of each system and related drainage basins. Plans for Phase 1 have been completed. Phase 2 will begin in the near future when funding permits.

TABLE 2

DRAINAGE SYSTEMS

PHASE	DRAINAGE SYSTEM	BASINS DRAINED BY EACH SYSTEM
1	1	1, 2
1	2	3, 4
1	3	5, 6, APS
2	4	7, 8, 9, 10, 11

A cross section of the street within each of the system limits was analyzed to determine if catch basins were necessary to meet the City requirements of maintaining one twelve (12) foot driving lane free of flowing or standing water in each traffic direction during the 10-year storm. The analysis did not show a need for the placement of catch basins based on this City criteria. However, catch basins were placed along Eubank Boulevard at the upstream side of intersections, in order to capture a portion of runoff before it crossed the intersecting street.

Phase 1 (Sta. 161+90.57 to Sta. 210+25.83)

Phase 1 incorporates improvements to Eubank Boulevard starting approximately at the entrance to Kirtland AFB to the south and ending approximately at the intersection of Southern Boulevard to the north.

System 1 (Sta. 161+90.78 to 164+86.89)

The proposed drainage structures for this system are a series of Type A catch basins to be located on each side of Eubank Boulevard (a total of five catch basins). The catch basins will discharge into an existing 90-inch storm drain. The proposed location for the connection of the catch basins and the 90-inch storm drain is an existing manhole. Because the manhole lies outside the southern limits of the Eubank Boulevard Improvements project, coordination

of the connection with the G Avenue reconstruction currently in design by Sandia National Laboratories will be required.

System 2 (Sta. 173+72.05 to 183+50.00)

The east side of System 2 will consist of three Type A catch basins connected to an existing storm drain. Two of the catch basins will be placed along Eubank Boulevard, and the third is to be placed at the low point in westbound Innovation Parkway.

The runoff from the south side of Innovation Parkway will flow into a catch basin that will be built with the development of the Sandia Science and Technology Park. This catch basin was not included in these proposed drainage improvements, because it will discharge to a new storm drain system that will not be built with the Eubank Boulevard Improvements.

Two Type A catch basins placed in series along the west side of Eubank Boulevard will discharge into a new 24-inch storm drain. This new storm drain will convey the flow across the roadway and into the existing 84-inch storm drain.

A Double Wing Type A catch basin is also proposed for the low area on the north side of Opportunity Avenue. A catch basin currently exists on the south side of Opportunity Avenue and discharges into an adjacent 84-inch storm drain. The proposed catch basin will connect to an existing 24-inch stub out.

System 3 (Sta. 199+38.47 to 206+09.82 on Eubank and Sta. 0+00 to 8+71.97 on Southern)

System 3 is an entirely stand-alone system that will eventually be part of the Gibson Corridor storm drain. As proposed, the east side of the system will consist of a series of Type A catch basins and a section of storm drain that will accept runoff from Eubank Boulevard and the APS property. A Type A catch basin will be placed south of the intersection of Eubank Boulevard and La Entrada Avenue and will be connected via an 18-inch storm drain to a Modified Double Wing Type A catch basin located at the low point of the roadway (Sta.

201+38.54). Runoff from the APS property will be collected within a depressed area and drained utilizing a ported riser connected with a 30" trunk line to the main 48" storm drain line. A modified storm drain manhole with an inlet cover at the start of the 48" line will collect runoff from the east side of Eubank Boulevard.

The 48-inch storm drain will also receive flow from a Double Wing Type A catch basin on the west side of Eubank Boulevard. As designed, the new storm drain will be able to convey the flow for fully developed conditions from the APS property. According to the Manzano Mesa Master Drainage Study, the APS site will be allowed to discharge at a maximum rate of 55 cfs under developed conditions and 28 cfs under existing conditions. The flow within the new storm drain will discharge into a temporary retention pond. Once the Gibson Corridor storm drain is constructed, the pond will no longer be necessary and will be removed.

The pond will be located west of the Eubank and Southern Boulevard intersection. This location was chosen because it will not have an impact on any future development that may occur within the Department of Energy (DOE) property. The proposed pond site is within the right-of-way that has been dedicated by DOE to the City of Albuquerque for the construction of the future Gibson Corridor Extension. Without the temporary retention pond, basins 5, 6 and the APS property would have no discharge point. Capacity constraints, elevation differences or, significant utility conflicts render connection to either system 2 or 4 unfeasible.

The required capacity of the temporary retention pond was determined by computing the volume of runoff entering the pond. The areas draining to the pond are street basins 5, 6 and the offsite property owned by the Albuquerque Public Schools (APS). Basins 5 and 6 were analyzed for fully developed conditions and the APS property was analyzed for existing conditions. Because there are no current plans for development of the APS property. It was also assumed that the Gibson Corridor storm drain would have been constructed before any development on the APS property has begun or ponding on the APS site would be required with the development of the site, to allow flows to be discharged at the historic (existing conditions) rate. The calculations were performed using the Excess Precipitation and

Volumetric Runoff (A.5) method in the DPM. The runoff volume was based on a 100-year, 10-day storm as required by City of Albuquerque design criteria for retention pond sizing. The runoff volumes from basins 5, 6 and the APS property are summarized in Table 1.

Phase 2

Phase 2 begins approximately at Southern Boulevard to the south and ends approximately at Central Avenue.

System 4 (Sta. 210+25.83 to 220+52.97)

Proposed improvements for the east side of Eubank Boulevard. include removing and replacing the existing Type A catch basins at the intersection of the north Costco Driveway and Eubank Boulevard. and at the intersection of the south Home Depot Driveway and Eubank Boulevard. The catch basins will be removed and replaced in order to accommodate the Eubank Boulevard. widening. Each of the new catch basins will discharge into the existing 54-inch storm drain as they currently do. A Type D catch basin is proposed to capture flow from the low point at Sta. 216+37.00 and convey the runoff north to a new catch basin and ultimately into the existing storm drain.

The west side of Eubank Boulevard. will be serviced by Type A catch basins discharging into a new 18-inch lateral storm drain and ultimately into the existing 54-inch storm drain. The 18-inch lateral will be connected to the 54-inch storm drain via an existing 18-inch storm drain. Because the connection is an 18-inch pipe, the proposed lateral storm drain will not be the typical minimum diameter of 24 inches. This was approved in a phone conversation with Brad Bingham of the City of Albuquerque. The phone conversation record is located in Appendix F. New Type A catch basins will be placed at the intersections of Eubank Boulevard. with Trumbull Avenue, Bell Avenue and Acoma Road.

The catch basins at the intersection of Eubank Boulevard. and Trumbull Avenue will capture all the flow draining to it and prevent runoff from flowing west along Trumbull Avenue.

A catch basin and valley gutter is proposed for the intersection of Eubank Boulevard. and Bell Avenue. The valley gutter will convey any residual flow north to the next catch basin along Eubank Boulevard at Acoma Road. The existing catch basin at this intersection must be removed and replaced with a Double Wing Type A catch basin to accommodate the sump condition in the area and the Eubank Boulevard. widening. Valley gutter and a 6-inch water block are also proposed at Acoma Road to ensure flow is not conveyed west along Acoma Road.

C. Hydraulic Analysis of Proposed Systems

The profiles of each proposed drainage system are located in Appendix D and the hydraulic grade line is displayed on each.

Systems 1-3 (Phase 1)

A hydraulic grade line analysis of each systems' proposed drainage structures indicates that surcharging will not occur at manhole and inlet locations under the given pressure flow and head-loss conditions.

Temporary Retention Pond

The temporary retention pond is a portion of system 3 that will retain all the runoff from the system-3 storm drain. Based on runoff values from basins 5, 6, and the APS property, the required capacity of the pond is 129,500 cubic feet (3.0 acre-feet). The available ponding depth will be 8 feet, not including the 1-foot of freeboard. The footprint area of the proposed pond will be approximately 0.8 acres.

Accumulated storm water within the temporary retention pond is required to be pumped since the pond must not store water longer than 192 hours (2 x 96 hours). The State Engineers Office normally requires retention ponds be drained/pumped within 96 hours or else water rights equaling the amount of stored water must be used. However, based on conversations with the State Engineers Office regarding this project they indicated the City

would be granted twice the normal period or 192 hours due to the temporary nature of the facility.

The depth within the pond resulting from a 2-year storm was determined to be 2.7' (32.5 inches). Within 192 hours, approximately 2 inches of water within the pond would evaporate and 24 inches would be lost due to infiltration for a total of 26 inches. Consequently for any storms slightly less than the 2-year storm or larger pumping will be required.

System 4 (Phase 2)

Based on the hydraulic grade line analysis of this system, surcharging will occur at multiple inlets and manholes. The two reasons for the surcharge occurring are: lack of capacity within the existing mainline storm drain and additional runoff proposed to be conveyed within the system.

Review of the as-builts for the existing 54-inch storm drain within Eubank Boulevard indicates that the existing storm drain does not have the capacity for the 100-year storm event. A copy of the as-builts for the existing 54-inch storm drain is included in Appendix E. The approved construction plans for the existing mainline storm drain show surcharge occurring at one manhole along the system and the hydraulic grade line at or above ground elevation at numerous locations. This lack of capacity will be further compounded by the proposed additional runoff to be conveyed.

This additional runoff arises from the City's plans to keep storm runoff from flowing west down Trumbull Avenue, Bell Avenue, Acoma Road and Cochiti Road as it currently does during storm events. Ensuring flow is not conveyed down these streets during typical storm events was the priority. Without increasing the size of the existing mainline storm drain, removing the possibility of surcharge occurring during the 100-year storm event is not feasible.

V. CONCLUSION

The hydraulic analysis of the Eubank Boulevard drainage improvements and the ultimate storm drain design, indicates that systems 1-3 have sufficient capacity for conveying runoff from the 100-year, 6-hour design storm. System 4 does not have the capacity to convey the 100-year storm event but will satisfy the priority of ensuring flow is not conveyed down the streets intersecting Eubank Boulevard on the west side of the roadway.

The floodplain adjacent to Eubank Boulevard, north of La Entrada Avenue will also be removed by the improvements proposed within this drainage report.

APPENDIX A

EXCERPTS FROM THE "CITY OF ALBUQUERQUE DEVELOPMENT PROCESS MANUAL"

SECTION 22.2, HYDROLOGY

(Chapter 22, Section 2)

of the

DEVELOPMENT PROCESS MANUAL

VOLUME 2, DESIGN CRITERIA
1997 REVISION

CITY OF ALBUQUERQUE, NEW MEXICO

Chapter 22, Section 2, Hydrology was prepared in cooperation with

BERNALILLO COUNTY, NEW MEXICO

and the

ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY

with the text prepared by the

D.P.M. DRAINAGE DESIGN CRITERIA COMMITTEE

The text of the 1997 revision was reformatted from the January 1993 revision. The equations, tabular values, examples and procedures in Chapter 22, Section 2, Hydrology were not changed by the 1997 revision.

PART A - PROCEDURE FOR 40 ACRE AND SMALLER BASINS

A simplified procedure for projects with sub-basins smaller than 40 acres has been developed based on initial abstraction/uniform infiltration precipitation losses and Rational Method procedures. For this procedure, Bernalillo County has been divided into four (4) Precipitation Zones.

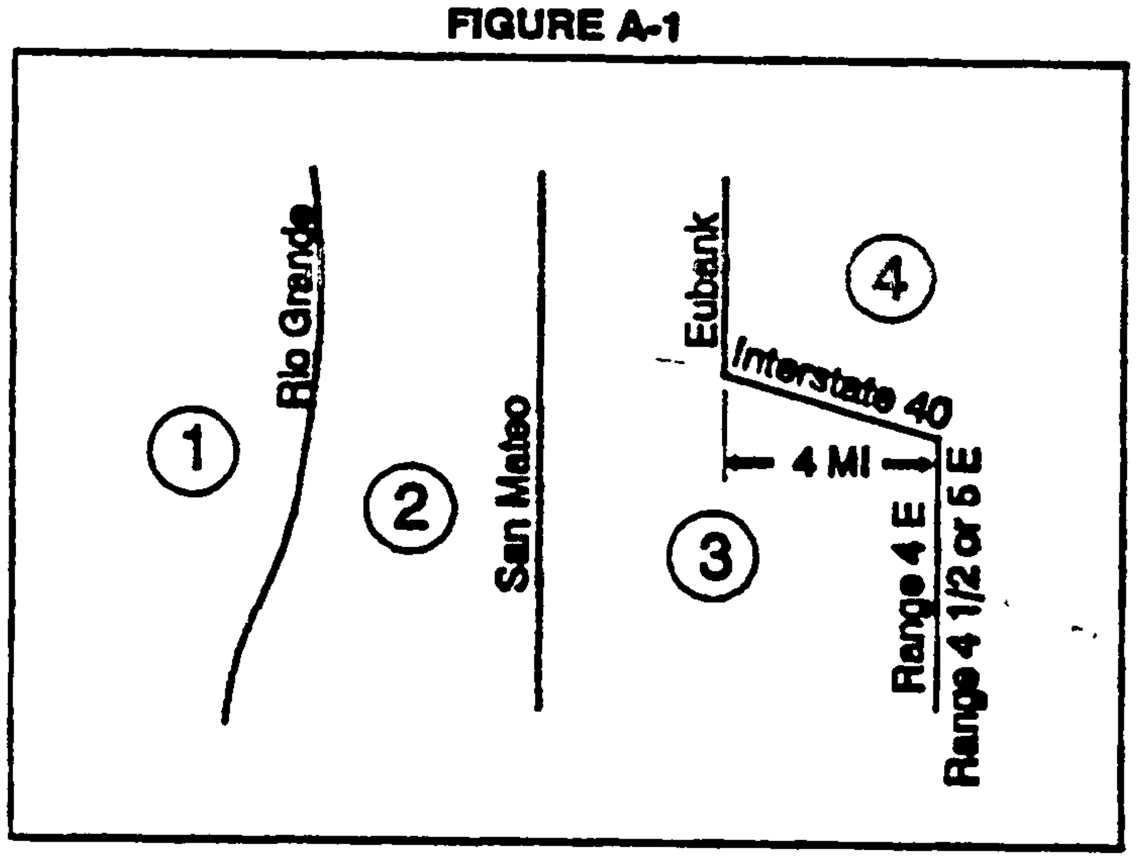
A.1 PRECIPITATION ZONES

Bernalillo County's four precipitation zones are indicated in TABLE A-1 and on FIGURE A-1.

TABLE A-1. PRECIPITATION ZONES

Zone	Location
1	West of the Rio Grande
2	Between the Rio Grande and San Mateo
3	Between San Mateo and Eubank, North of Interstate 40; and between San Mateo and the East boundary of Range 4 East, South of Interstate 40
4	East of Eubank, North of Interstate 40; and East of the East boundary of Range 4 East, South of Interstate 40

FIGURE A-1



Where a watershed extends across a zone boundary, use the zone which contains the largest portion of the watershed.

EXAMPLE A-4

Find the 100-year 24-hour and 4-day runoff volume, V_{1440} and V_{4days}, for the area in Example A-3.

V_{360} = 2.41 acre-feet

V_{1440} = 2.41 + 7 ac * (2.66 - 2.20) / 12 = 2.68 acre-feet

V_{4DAYS} = 2.41 + 7 ac * (3.12 - 2.20) / 12 = 2.95 acre-feet

A.6 PEAK DISCHARGE RATE FOR SMALL WATERSHEDS

Small watersheds are less than or equal to 40 acres.

Peak Discharge

Using a 0.2-hour (12-minute) time of concentration, peak discharge, Q_P, per acre is the volume of excess precipitation in the heaviest 12-minute portion of the storm, divided by the time increment 12 minutes, and multiplied by an attenuation factor. The attenuation factor (0.59 for treatment A, 0.67 for treatment B, 0.75 for treatment C and 0.93 for treatment D) describes the effect of routing. Determine the peak discharge using the values in TABLE A-9, which have been adjusted to consider the effects of initial abstraction.

TABLE A-9. PEAK DISCHARGE (CFS/ACRE)

Zone	Treatment		100-YR [2-YR, 10-YR]	
	A	B	C	D
1	1.29 [0.00, 0.24]	2.03 [0.03, 0.76]	2.87 [0.47, 1.49]	4.37 [1.69, 2.89]
2	1.56 [0.00, 0.38]	2.28 [0.08, 0.95]	3.14 [0.60, 1.71]	4.70 [1.86, 3.14]
3	1.87 [0.00, 0.58]	2.60 [0.21,1.19]	3.45 [0.78, 2.00]	5.02 [2.04, 3.39]
4	2.20 [0.05, 0.87]	2.92 [0.38, 1.45]	3.73 [1.00, 2.26]	5.25 [2.17, 3.57]

TABLE A-4. LAND TREATMENTS	
Treatment	**Land Condition**
A	Soil uncompacted by human activity with 0 to 10 percent slopes. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, groundcover and infiltration capacity.
B	Irrigated lawns, parks and golf courses with 0 to 10 percent slopes. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes greater than 10 percent and less than 20 percent.
C	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Gravel or rock on plastic (desert landscaping). Irrigated lawns and parks with slopes greater than 10 percent. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes at 20 percent or greater. Native grass, weed and shrub areas with clay or clay loam soils and other soils of very low permeability as classified by SCS Hydrologic Soil Group D.
D	Impervious areas, pavement and roofs.
Most watersheds contain a mix of land treatments. To determine proportional treatments, measure respective subareas. In lieu of specific measurement for treatment D, the areal percentages in TABLE A-5 may be employed.	

To determine the peak rate of discharge,

1) Determine the area in each treatment, A_A, A_B, A_C and A_D.

2) Multiply the peak rate for each treatment by the respective areas and sum to compute the total Q_P.

$$\text{Total } Q_P = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D \qquad \text{(a-10)}$$

Example A-5

Find 100-year Q_P for 14 acres in zone 1. The four land treatments are: 3 acres in treatment A, 5 acres in treatment B, 2 acres in treatment C and 4 acres in treatment D.

Total Q_P = (1.29 * 3) + (2.03 * 5) + (2.87 * 2) + (4.37 * 4) = 37.24 cfs

3) Approximately the same results can be achieved by a Rational Method solution. The 0.2-hour (12-minute) peak intensities, I, are given in TABLE A-10 and Rational Method coefficients, C, are given in TABLE A-11.

$$\text{Total } Q_P = (C_A * I * A_A) + (C_B * I * A_B)$$
$$+ (C_C * I * A_C) + (C_D * I * A_D) \qquad \text{(a-11)}$$

TABLE A-10. PEAK INTENSITY (IN/HR at t_c = 0.2 hour)

Zone	Intensity	100-YR [2-YR, 10-YR]
1	4.70 [1.84, 3.14]	
2	5.05 [2.04, 3.41]	
3	5.38 [2.21, 3.65]	
4	5.61 [2.34, 3.83]	

TABLE A-6. INITIAL ABSTRACTION (IA)	
Treatment	**Initial Abstraction (inches)**
A	0.65
B	0.50
C	0.35
D	0.10

Infiltration is the only significant abstraction after the initial abstraction. After initial abstraction is satisfied, treat infiltration as a constant loss rate as specified in TABLE A-7.

TABLE A-7. INFILTRATION (INF)	
Treatment	**Loss Rate (inches/hour)**
A	1.67
B	1.25
C	0.83
D	0.04*
* Treatment D infiltration rate is applicable from 0 to 3 hours; use uniform reduction from 3 to 6 hours, with no infiltration after 6 hours.	

Runoff from a previous event can saturate a channel bed, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

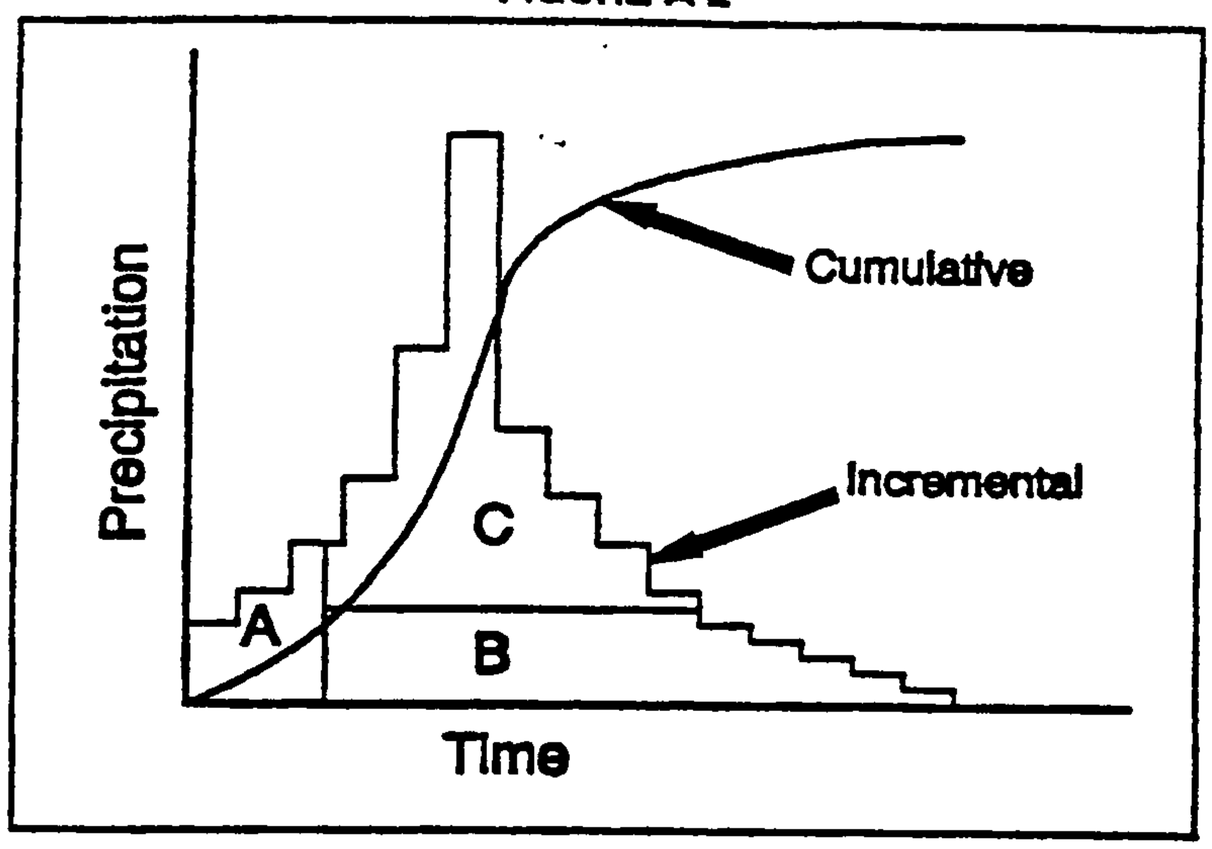
A.5 EXCESS PRECIPITATION & VOLUMETRIC RUNOFF

USED FOR POND SIZING

Excess precipitation, E, is the depth of precipitation remaining after abstractions are removed. Excess precipitation does not depend on watershed area. Excess precipitation is determined by subtracting the initial abstraction and infiltration from the design storm hydrograph. FIGURE A-2 illustrates the development of excess precipitation. The curved line plots cumulative precipitation. Precipitation intensities (in/hr) are shown as a histogram. Initial abstraction is area A. The horizontal line is at a height corresponding to the infiltration rate. Infiltration loss is area B. The remaining histogram, area C, is excess precipitation.

"POND SIZING"

FIGURE A-2



Excess precipitation, E, by zone and treatment is summarized in TABLE A-8.

(NOTE: In this table and several tables which follow, corresponding values for 2- and 10- year storms are shown in brackets below each 100-year value)

TABLE A-8: EXCESS PRECIPITATION, E (INCHES) - 6 HOUR STORM

Zone	Treatment 100-YR [2-YR, 10-YR]			
	A	B	C	D
1	0.44 [0.00, 0.08]	0.67 [0.01, 0.22]	0.99 [0.12, 0.44]	1.97 [0.72, 1.24]
2	0.53 [0.00, 0.13]	0.78 [0.02, 0.28]	1.13 [0.15, 0.52]	2.12 [0.79, 1.34]
3	0.66 [0.00, 0.19]	0.92 [0.06, 0.36]	1.29 [0.20, 0.62]	2.36 [0.89, 1.50]

"Pond Sizing"

To determine the volume of runoff,

1) Determine the area in each treatment, A_A, A_B, A_C, A_D

2) Compute the weighted excess precipitation, E

$$\text{Weighted E} = \frac{E_A A_A + E_B A_B + E_C A_C + E_D A_D}{A_A + A_B + A_C + A_D} \qquad \text{(a-5)}$$

3) Multiply the weighted E by the watershed area.

$$V_{360} \text{ (as volume)} = \text{weighted E} * (A_A + A_B + A_C + A_D) \qquad \text{(a-6)}$$

EXAMPLE A-3
Find the 100-year V_{360} for 30 acres in zone 1. Eight acres are treatment A, 10 acres are treatment B, 5 acres are treatment C, and 7 acres are treatment D.
Weighted E = ((8 * 0.44) + (10 * 0.67) + (5 * 0.99) + (7 * 1.97)) / 30 = 0.965 inches
Volume = (0.965 * 30) / 12 = 2.41 acre-ft. = V_{360}

For ponds which hold water for longer than 6 hours, longer duration storms are required to establish runoff volumes. Since the additional precipitation is assumed to occur over a long period, the additional volume is based on the runoff from the impervious areas only.

For 24-hour storms:

$$V_{1440} = V_{360} + A_D * (P_{1440} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-7)}$$

For 4-day storms:

$$V_{4DAYS} = V_{360} + A_D * (P_{4DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-8)}$$

STORM FOR RETENTION POND SIZING

For 10-day storms:

$$V_{10DAYS} = V_{360} + A_D * (P_{10DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-9)}$$

"POND SIZING"

"Type B" basins are generally placed downstream of and/or in conjunction with "Type A" basins on streets other than arterials and collectors. This type basin has potential to collect substantial runoff when the grating is clean. If "Type B" basins are used alone, without a "Type A" within 150 feet upstream, the capacity shown in Plate 22.3 D-7 should be reduced 15% due to clogging. "Type B" basins are used with standard 8" curb and gutter.

"Type C" basins are generally placed downstream of and/or in conjunction with "Type A" basins. If "Type C" basins are used without a "Type A" within 150 feet upstream, the capacity shown in Plates 22.3 D-5 and 22.3 D-6 should be reduced 15% for clogging. "Type C" basins are used with standard 8" curb and gutter.

"Type D" basins are generally used on streets with slope greater than 5%, in driveways and in certain valley areas as described above. "Type D" basins can be used with either standard 8" curb and gutter or with mountable curb.

The number of catch basins to be connected in series should not exceed two. If the connection of more than two catch basins in series is unavoidable, consideration should be given to designing a lateral drain.

c. Connector Pipe
   (1) The minimum diameter of connector pipe is 18 inches.
   (2) The horizontal alignment of connector pipes must not contain angle points or bends, unless approved by the City Engineer.
   (3) Connector pipes outletting into a pipe from both sides of a street should be offset 8 feet or more at the main line. Exceptions to this criterion must be approved by the City Engineer. Connections at manholes or junction structures are preferred.
   (4) ~~The minimum length of connector pipe to be installed between catch basins in series shall be 12 feet, unless otherwise approved by the City Engineer.~~ The catch basin spacing shall be a minimum of 25 feet distance between curb transitions.
   (5) Catch basin connector pipes shall outlet at the downstream end of the catch basins, unless prevented by field conditions. Downstream, in this paragraph, refers to the directions of the gutter slope at the catch basin in question.
   (6) Where feasible, connector pipes should be located so as to avoid, as much as possible, cutting into existing cross gutters and spandrels.
   (7) The conversions of "type A's, B's or C's to D's basins will not be permitted. If the catch basin is in conflict with a driveway, the catch basin will be removed and replaced with another inlet outside of the driveway. To avoid conflicts with driveways, the engineer should identify the proposed driveways on the grading plan when catch basins front the lots.

E. <u>Street Hydraulics</u>

1. A secondary use of the street network is the conveyance of storm runoff. This secondary use must always be subsidiary to the primary function of streets which is the safe conveyance of people and vehicles. The goals of street hydraulic design are therefore:
   a. To provide an economical means of transporting storm runoff.
   b. To ensure that the safety and convenience of the public are preserved.
   c. To prevent storm runoff, once collected by the street system, from leaving the street right-of-way except at specially designated locations.

2. Street hydraulic design critical are as follows:

a. Manning's roughness coefficient is 0.017.
b. Flow depths ~~plus velocity head~~ ($v^2/2g$)in the event of the 100-year design discharge may not exceed 0.2 feet above curb height ~~or 0.87 feet at any location~~ and ~~shall be contained within the street right-of-way~~.
c. Flow depths in the event of the 10-year design discharge may not exceed 0.5 feet in any collector or arterial street. One ~~twelve (12) foot driving~~ lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.
d. The product of depth times velocity shall not exceed 6.5 in any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet.)

3. For streets with more than two driving lanes in each direction:

   a. The product of depth times velocity may not exceed 6.5 at any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet).
   b. Inverted crown streets are prohibited.
   c. The assumption of equal flow distribution between gutters on undivided streets and between street sections on divided streets is only valid where its validity can be demonstrated.

4. Plates 22.3 D-1 through 22.3 D-4 may be used where applicable in the hydraulic design of streets. T-intersections, radical slope changes and intersections are potential locations for hydraulic jumps when upstream slopes are steeper than critical slope.

   When conditions indicate that a hydraulic jump or that the effects of superelevation will allow runoff to exceed street hydraulic design criteria, provisions must be made to for treatment of the problem. The warping of street sections and the construction of deflector walls for these purposes is prohibited unless specifically authorized by the City Engineer.

5. Intersections and other radical changes in street cross section and slope require special consideration whenever the flow depth/street slope relationship results in flows occurring in the supercritical flow regime. The critical slope line shown on the street rating curves is used to determine on which side of critical depth the flow occurs and if slope or cross section changes will allow the flow to cross through critical depth from supercritical.

   If flow is likely to cross into the subcritical flow range, then Plate 22.3 E-1, "Tail Water vs. Froude Number" is used to determine the height and Plate 22.3 E-2, "Length of Jump vs. Froude Number" figure is used to determine jump length. The height of jump should not exceed ~~0.2 feet above curb height~~ ~~0.87 feet~~ and ~~the jump should~~ ~~shall~~ be contained within the street right-of-way.

APPENDIX B

PLATES

PLATE 1 – DRAINAGE BASIN MAP
PLATE 2 – DRAINAGE BASIN MAP

APPENDIX C

CALCULATIONS

System 1 – Sta: 161+90.78 to 164+86.89

System 2 – Sta: 173+72.05 to 183+50

System 3 – Sta: 199+38.47 to 206+09.82 on Eubank
Sta: 0+00 to 8+71.97 on Southern

System 4 – Sta: 210+25.83 to 220+52.97

SYSTEM 1 HYDROLOGY CALCS:

- BASIN #2 (WEST SIDE):

A = 1.82 ACRES

100% "D"

Q = 1.82 * 5.02 = 9.1 cfs

- BASIN #1 (EAST SIDE):

A = 3.64 ACRES

90% "D"

10% "C"

Q = (0.9 * 3.64 * 5.02) + (0.1 * 3.64 * 3.45) = 17.7 cfs

SYSTEM 2

"SYSTEM 2" DRAINAGE BASIN CALCULATIONS:

- **BASIN 3**

AREA = 3.15 ACRES

LAND TREATMENT TYPES: 100% "D"

$Q_{100\text{-}YR}$ = 3.15 * 5.02 = 15.8 cfs

- **BASIN 4**

AREA = 3.21 ACRES

LAND TREATMENT TYPES: 100% "D"

$Q_{100\text{-}YR}$ = 3.21 * 5.02 = 16.1 cfs

"System 3" Drainage Basin Calculations

- **Basin 5**

Area = 2.84 acres
Land treatment types: 100% "D"

$Q_{100\text{-}yr} = 2.84 * 5.02 = \underline{14.3\ cfs}$

- Basin 5 Runoff volume:

Excess precipitation = $E = \frac{2.84 * 2.36}{2.84} = 2.36$ inches

$V_{360} = \frac{2.36}{12} * (2.84 * 43{,}560) = 24{,}330\ ft^3 = 0.56$ acre-feet

$V_{10\text{-}days} = 0.56 + [2.84 * (4.90 - 2.60)/12] = 1.10$ acre-feet $= 48{,}040\ ft^3$

- **Basin 6**

Area = 2.66 acres
Land treatment types: 100% "D"

$Q_{100\text{-}yr} = 2.66 * 5.02 = \underline{13.4\ cfs}$

- Basin 5 Runoff volume:

Excess precipitation = $E = 2.36$ inches

$V_{360} = \frac{2.36}{12} * (2.66 * 43{,}560) = 22{,}790\ ft^3 = 0.52$ acre-feet

$V_{10\text{-}days} = 0.52 + [2.66 * (4.90 - 2.60)/12] = 1.03$ acre-feet $= 44{,}870\ ft^3$

SYSTEM 3

· APS PROPERTY (UNDEVELOPED)

AREA = 13.3 ACRES

LAND TREATMENT TYPES : 85% "A"
15% "C"

$Q_{100\text{-}YR} = (0.85 * 13.3 * 1.87) + (0.15 * 13.3 * 3.45) = \underline{28.0 \text{ cfs}}$

· APS PROPERTY (DEVELOPED)

LAND TREATMENT TYPES : 25% "B"
25% "C"
50% "D"

$Q_{100\text{-}YR} = (0.25 * 13.3 * 2.6) + (0.25 * 13.3 * 3.45) + (0.5 * 13.3 * 5.02) = 53.5$ CFS

USE $\underline{55 \text{ CFS}}$

· APS PROPERTY RUNOFF VOLUME (UNDEVELOPED)

EXCESS PRECIPITATION = E

$$\text{WEIGHTED } E = \frac{(13.3 * 0.85 * 0.66) + (13.3 * 0.15 * 1.29)}{13.3} = 0.755 \text{ INCHES}$$

$$V_{360} = \frac{0.755}{12} * (13.3 * 43{,}560) = 36{,}450 \text{ FT}^3 = 0.84 \text{ ACRE-FEET}$$

$V_{360} = V_{10\text{-}DAYS}$ BECAUSE THE UNDEVELOPED APS PROPERTY HAS NO LAND TREATMENT OF TYPE "D".

Bohannan Huston

PROJECT NAME _____ SHEET _____ OF _____
PROJECT NO _____ BY _____ DATE _____
SUBJECT System 3 CH'D _____ DATE _____

SYSTEM 5

"SYSTEM 3" POND SIZING

ASSUMPTIONS: · APS PROPERTY UNDEVELOPED (13.3 ACRES)
· EUBANK ROADWAY FULLY DEVELOPED (5.5 ACRES)

TREATMENT TYPES:

· APS: 85% "A"
15% "C"

· EUBANK ROADWAY: 100% "D"

EXCESS PRECIPITATION = E

$$\text{WEIGHTED } E = \frac{(0.66 * 13.3 * 0.85) + (1.29 * 13.3 * 0.15) + (2.36 * 5.5)}{18.8}$$

= 1.22 INCHES OF EXCESS PRECIPITATION

$$V_{360} = \frac{1.22}{12} * (18.8 \times 43{,}560) = 83{,}260 \text{ ft}^3 = 1.91 \text{ ACRE FEET}$$

$$V_{10\text{-DAYS}} = 1.91 + [5.5 * (4.90 - 2.60)/12] = 2.97 \text{ ACRE-FEET} = \underline{\underline{129{,}375 \text{ FT}^3}}$$

TEMPORARY RETENTION POND CAPACITY USE 129,500 FT^3

Bohannan Huston

PROJECT NAME _____ SHEET _____ OF _____
PROJECT NO. _____ BY _____ DATE _____
SUBJECT SYSTEM 3 _____ CH'D _____ DATE _____

Capacity of ported riser

Eubank Expansion - APS POND
PORTED RCP RISER TOWER DESIGN

DESIGN PARAMATERS
Peak Discharge Rate at 100 year water surface =
Peak Discharge Rate at PMP water surface =
Outlet Works to be ported riser tower
Riser tower invert =
Top of Pond = 5069.45
Orifice Coefficient = 0 61

2 rows of 6, 6" openings
rows 1' apart, center-to-center

36" Riser

PORTED RCP RISER WITH TOP OPENING

	WATER ELEVATION (ft)	OPENING ROW NUMBER	NUMBER OF OPENINGS	OPENING DIAMETER (in)	OPENING AREA (sf)	ROW #1 OPENING HEAD (ft)	ROW #2 OPENING HEAD (ft)	ROW #3 OPENING HEAD (ft)	ROW #4 OPENING HEAD (ft)	ROW #5 OPENING HEAD (ft)	CUM. PORT FLOW RATE (cfs)	TOWER TOP OPENING WEIR LENGTH (ft)	WEIR HEAD (ft)	WEIR FLOW RATE (cfs)	TOTAL FLOW RATE (cfs)	ADJUSTED FLOW RATE (cfs)
											0.00			0.00	0 00	0.00
POND INVERT	5466	1	6	6	1.18	0 00	0.00	0.00	0.00	0.00	0.00			0.00	0 00	0.00
	5467	2	6	6	1.18	0.50	0.00	0.00	0.00	0.00	4.08			0.00	4.08	2 04
TOP OF TOWER	5468	3	0	4	0.00	1.50	0.50	0 00	0.00	0 00	11.14	9.42	0	0.00	11.14	5.57
	5469		0	8	0.00	2.50	1.50	1.00	0.00	0.00	16.18		1	24.77	40.96	20.48
	5469		0	8	0.00	2.50	1.50	1.00	1.00	0.00	16.18		1	24 77	40.96	20.48
TOP OF POND	5469 45					2 95	1.95	1 45	1.45	0.45	17.96		1.45	43 26	61.22	30.61

28.27433

30.61 > 28 CFS ✓

Capacity of outflow pipe for APS Pond/Ported Riser

"Calculated Values"

Pipes

Orifice diameter (in.)	=	30								
Area (ft^2)	=	4.91								
C	=	0.6								
Water surface elevation (ft)	=	0	1	2	3	4	5	5.24		
Headwater (ft)	=	0.00	0.00	0.75	1.75	2.75	3.75	3.99	0.00	0.00	0.00
Q (ft^3/s)	=	0.0	0.0	20.5	31.3	39.2	45.8	47.2	0.0	0.0	0.0
Number of Barrells	1	0.0	0.0	20.5	31.3	39.2	45.8	47.2	0.0	0.0	0.0

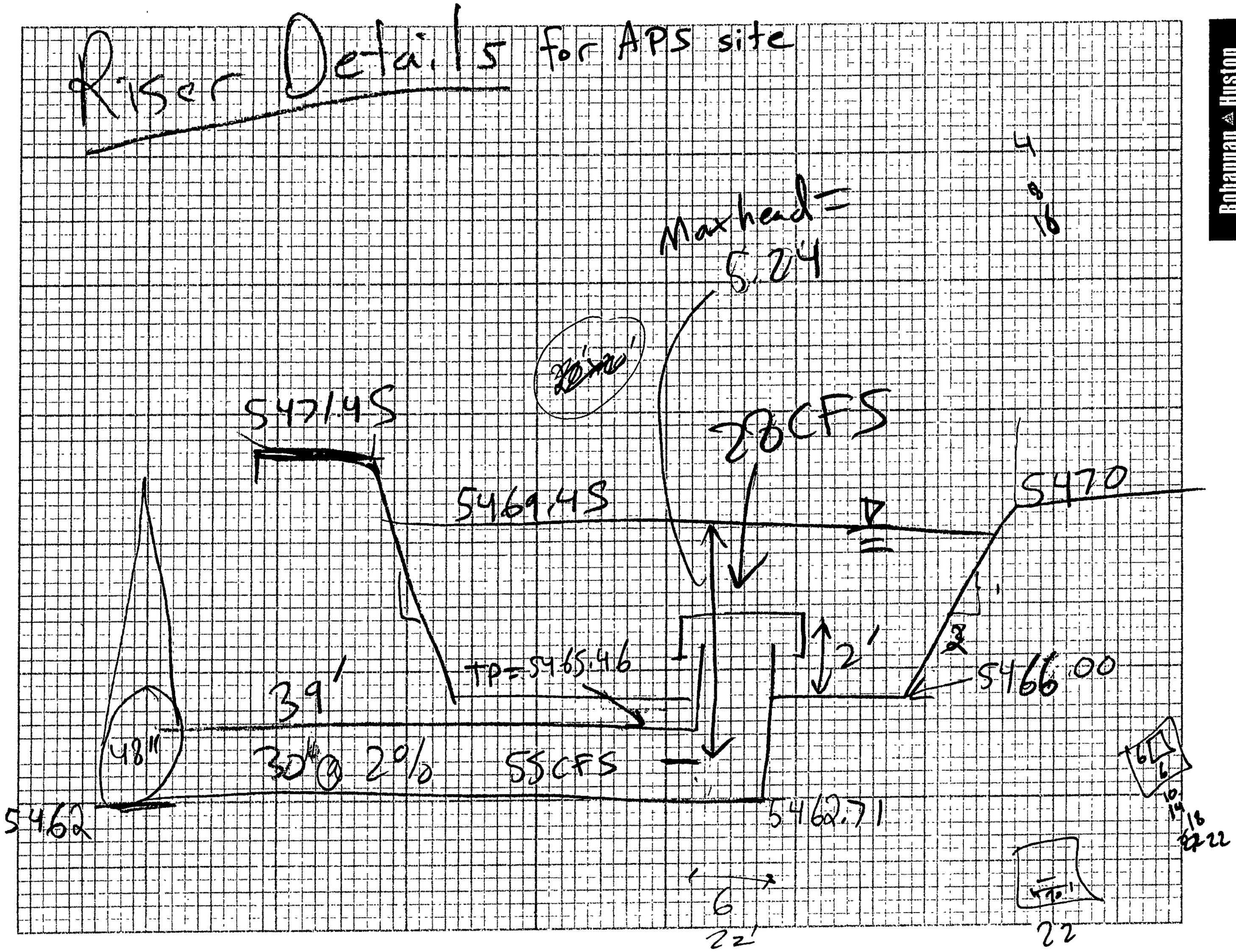
47.2 > 28.0 CFS ✓

Capacity of 30" = 62.39 CFS > 55 CFS ✓

with n = .013

s = 2%

Riser Details for APS site



Bohannan Huston

ENGINEERS PLANNERS PHOTOGRAMMETRISTS
SURVEYORS SOFTWARE DEVELOPERS

PROJECT NAME _____ SHEET _____ OF _____

PROJECT NO. _____ BY _____ DATE _____

SUBJECT _____ CH'D _____ DATE _____

Pond @ Eubank and Southern

Infiltration (See Attached)

$= 3 \times 10^{-6}$ ft/s * 96 hrs = 1.04'

Evaporation = 7'/year (from State engineer off.)

= .08' in 96 hrs ≈ 1"

Total = 1.12'/96 hrs ≈ 13"/96 hrs

2yr - 24hr storm produces approx. 3' deep accumulation in pond.

Pond is 8' deep.

Bohannan Huston

ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

PROJECT NAME Eubank - Pond SHEET ____ OF ____

PROJECT NO. 00268 BY GMM DATE 6/19/02

SUBJECT Pond Calculations CH'D ____ DATE ____

Runoff volume in pond resulting from a 2-yr 24-hr storm:

HYDROGRAPH FROM AREA PONDIN

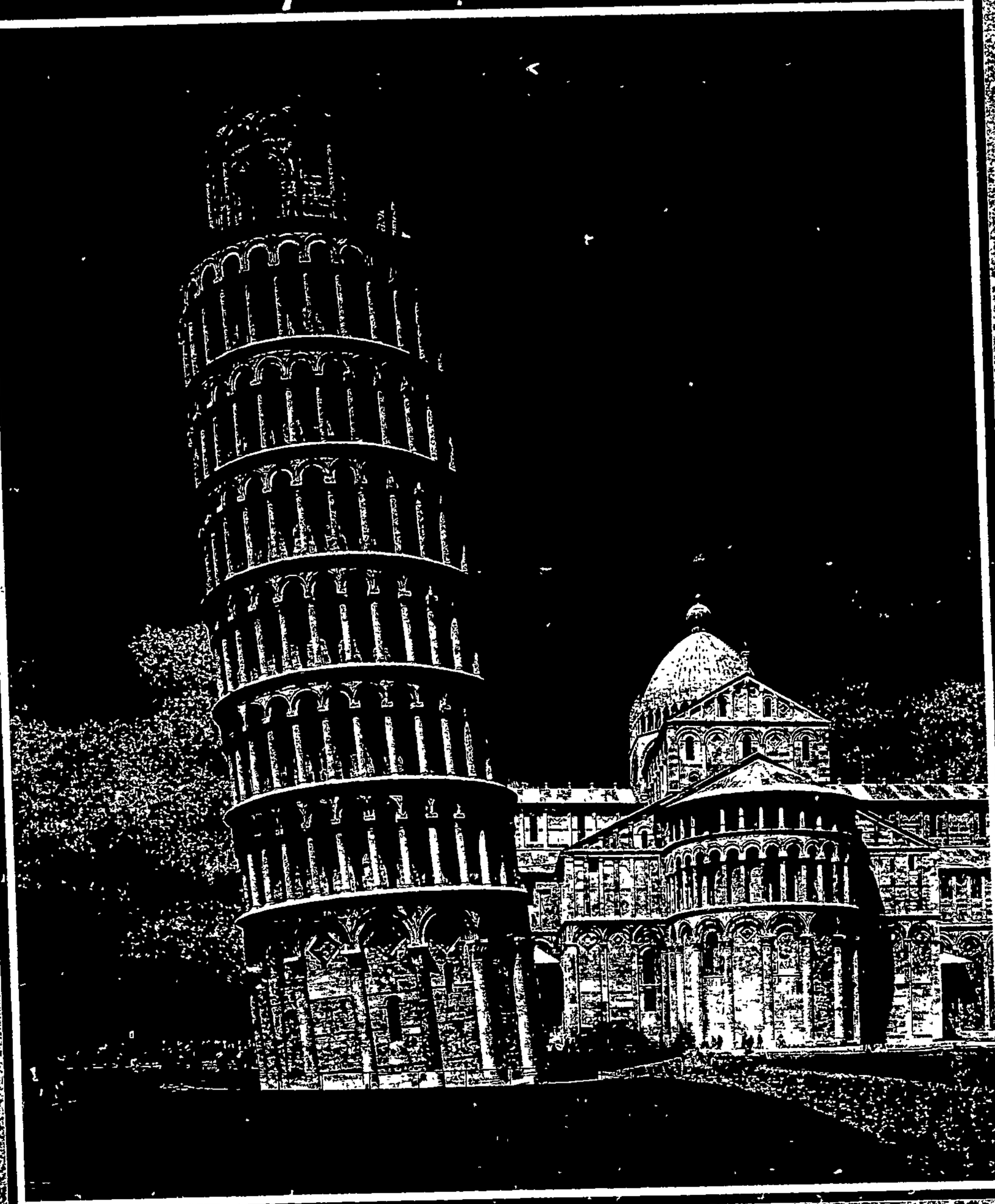
TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS
.000	.0	16.500	.1	5.500	.1	22.000	.1
.500	.0	17.000	.1	6.000	.1	22.500	.1
1.000	.0	17.500	.1	6.500	.1	23.000	.1
1.500	10.7	18.000	.1	7.000	.1	23.500	.1
2.000	3.0	18.500	.1	7.500	.1	24.000	.1
2.500	.4	19.000	.1	8.000	.1	24.500	.0
3.000	.1	19.500	.1	8.500	.1	25.000	.0
3.500	.0	20.000	.1	9.000	.1	9.500	.1
4.000	.0	20.500	.1	10.000	.1	10.500	.1
4.500	.0	21.000	.1	11.000	.1	11.500	.1
5.000	.1	21.500	.1	12.000	.1	12.500	.1
13.000	.1	13.500	.1	14.000	.1	14.500	.1
15.000	.1	15.500	.1	16.000	.1		

RUNOFF VOLUME = .33713 INCHES=.5286 ACRE-FEET=23026 cu. ft.

PEAK DISCHARGE RATE = 11.35 CFS AT 1.550 HOURS BASIN AREA = .0294 SQ. MI.

GEOTECHNICAL ENGINEERING

Principles and Practices



DONALD P. CODUTO

Q also will be 1,000,000 times smaller. The low k in clays is due to the small particle size (and therefore small void size). It is not due to water being absorbed by the clay.

TABLE 7.1 TYPICAL VALUES OF HYDRAULIC CONDUCTIVITY, k, FOR SATURATED SOILS

Soil Description	Hydraulic Conductivity, k	
	(cm/s)	(ft/s)
Clean gravel	1 to 100	3×10^{-2} to 3
Sand-gravel mixtures	10^{-2} to 10	3×10^{-4} to 0.3
Clean coarse sand	10^{-2} to 1	3×10^{-4} to 3×10^{-2}
Fine sand	10^{-3} to 10^{-1}	3×10^{-5} to 3×10^{-3}
Silty sand	10^{-3} to 10^{-2}	3×10^{-5} to 3×10^{-4}
Clayey sand	10^{-4} to 10^{-2}	3×10^{-6} to 3×10^{-4}
Silt	10^{-8} to 10^{-3}	3×10^{-10} to 3×10^{-5}
Clay	10^{-10} to 10^{-6}	3×10^{-12} to 3×10^{-8}

Geotechnical engineers also use the term *coefficient of permeability* to describe k. Unfortunately, this term can generate some confusion because it has at least two definitions:

- The *coefficient of permeability*, k, in Darcy's Law describes the ease with which a certain liquid flows through a certain soil. It depends on both the soil and the liquid, has units of L/t, and is the same as *hydraulic conductivity*.
- The *intrinsic permeability* (also called the *specific permeability*) which depends only on the soil. It has units of L^2. This definition is used primarily by hydrogeologists and petroleum geologists.

Both parameters are frequently called "permeability" and the variables K and k have been used for both. In addition, the intrinsic permeability is sometimes measured with a special unit called Darcys, even though it is not the proper parameter for use in Equation 7.8. These inconsistencies in terminology can be a source of confusion. Whenever in doubt, check the units to determine which "permeability" is being used.

To avoid confusion, geotechnical engineers are gradually dropping the term "coefficient of permeability" and using "hydraulic conductivity" instead.

Hydraulic Conductivity Tests (Permeability Tests)

Various laboratory and in-situ tests are available to measure the hydraulic conductivity. However, the results from both types are often judged with some skepticism because we are not sure if the test samples are truly representative. Even small differences in the soil classification can make a big difference in k, as illustrated in Table 7.1. Thus, it is good to perform many tests and review the scatter in the results.

Ano
represent
tests are b
Eve
order of =
significan
conductiv

Co

The *cons*
head of w

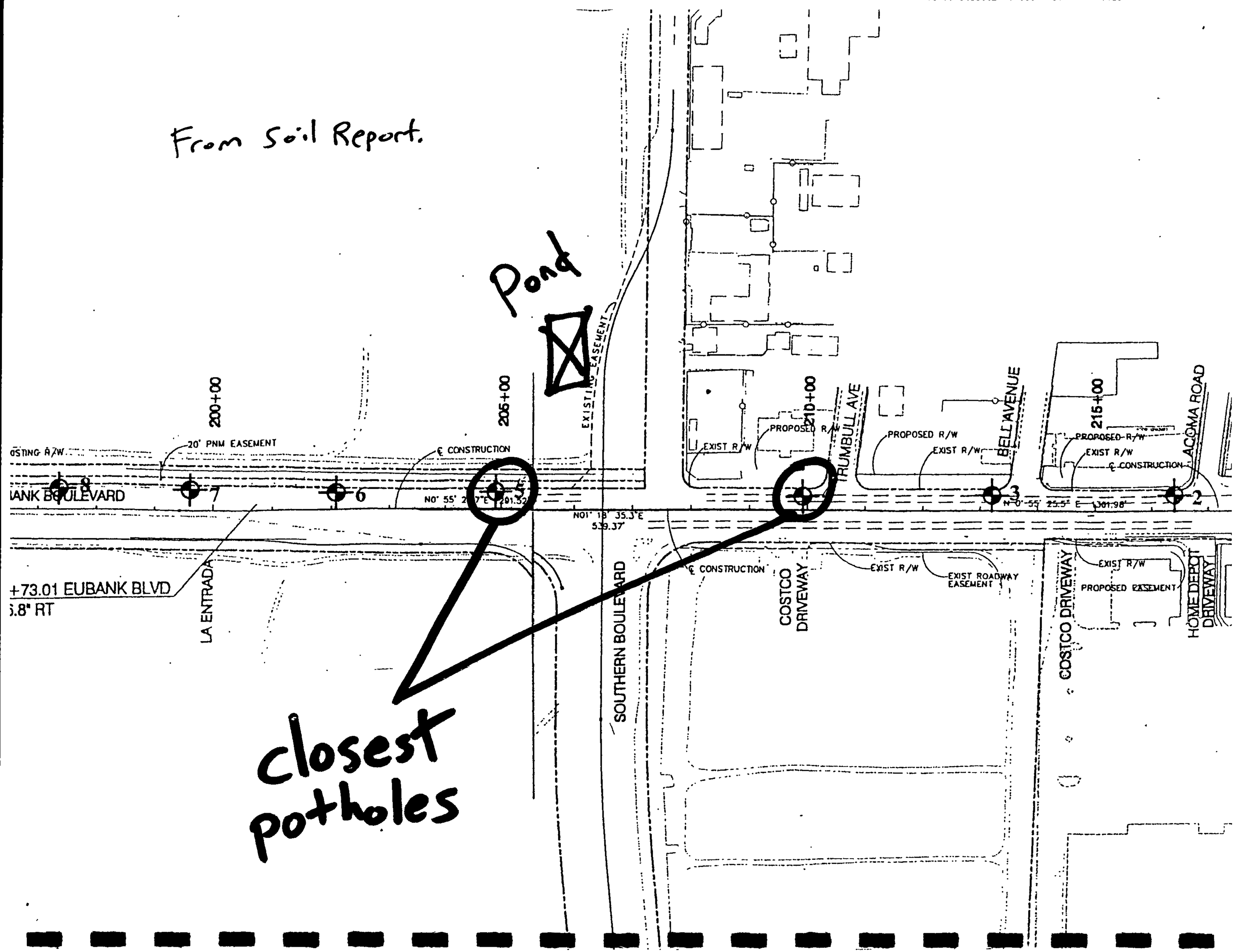
Figure 7.11 A co
constant heads. Th

Pond depth resulting from a 2- yr 24-hr storm:

Stage Storage Calculations (Average End Area Method)

Pond Title
Job No. 2 year storm requires 23026 cu. Ft. (reaches depth of approx. 2.7 ft

ELEV. (FT)	AREA (AC)	AVG. AREA (AC)	HEIGHT (FT)	VOLUME (AC-FT)	TOTAL VOLUME (AC-FT)	TOTAL VOLUME (CU-FT)
2907	5453	0.066736			0.0	0
		0.116793	1	0.1		0
7268	5454	0.16685			0.1	5087.5
		0.212121	1	0.2		0
11212	5455	0.257392			0.3	14327.5
		0.280108	1	0.3		0
13191	5456	0.302824			0.6	26529
		0.326194	1	0.3		0
15227	5457	0.349564			0.9	40738
		0.373577	1	0.4		0
17319	5458	0.39759			1.3	57011
		0.422257	1	0.4		0
19468	5459	0.446924			1.7	75404.5
		0.472234	1	0.5		0
21673	5460	0.497544			2.2	95975
		0.523508	1	0.5		0
23935	5461	0.549472			2.7	118779
		0.57609	1	0.6		0
26254	5462	0.602709			[redacted]	143873.5
		0.600023	1	0.6		0
26020	5463	0.597337			3.9	170010.5
		0.529706	1	0.5		0
20128	5464	0.462075			4.4	193084.5
		0.401653	1	0.4		0
14864	5465	0.34123			4.8	210580.5
		0.275253	1	0.3		0
9116	5466	0.209275			5.1	222570.5
		0.143974	1	0.1		0
3427	5467	0.078673			5.3	228842


From Soil Report.
Pond
closest potholes
200+00
205+00
210+00
215+00
20' PNM EASEMENT
℄ CONSTRUCTION
EXISTING R/W
EXISTING EASEMENT
PROPOSED R/W
EXIST R/W
TRUMBULL AVE
BELL AVENUE
ACOMA ROAD
PROPOSED R/W
EXIST R/W
℄ CONSTRUCTION
N0° 55' 25.5" E 301.98'
N01° 18' 35.3"E
539.37'
+73.01 EUBANK BLVD
3.8" RT
LA ENTRADA
SOUTHERN BOULEVARD
℄ CONSTRUCTION
COSTCO DRIVEWAY
EXIST R/W
EXIST ROADWAY EASEMENT
COSTCO DRIVEWAY
EXIST R/W
PROPOSED EASEMENT
HOME DEPOT DRIVEWAY

V & A

LOG OF TEST HOLE NO. 4

From soil report

Project: Reconstruction of Eubank Boulevard
Elevation - Top of Test Hole: N/A
Depth to Groundwater: Not Encountered

Project No.: 01-1-259
Date Drilled: 8/4/2001
Method: 6" HSA

Depth, feet	Blows/Foot	Sample Type	Dry Density pcf	Water Content, %	Additional Testing	Unified Classification	Material Description
							8" asphalt
1						SM	SAND, very silty, fine to coarse grained, gravelly, medium dense, medium moist, light brown
2	29	S		3.5	1		
3							
4						GM	GRAVEL, silty, very sandy, fine to coarse grained, medium dense, medium moist, light brown to grey
5							
6	5	S		2.3	1	SM	SAND, silty, fine to coarse grained, gravelly, loose, moist, brown
7							
8							
9						CL	CLAY, sandy, fine grained, medium stiff, medium moist, brown
10							
11	8	S		8.2			
12							Bottom of hole at 11½'

ADDITIONAL TESTS: 1= Sieve Analysis 2= Atterberg Limits 3=Direct Shear 4=R-Value 5=Other

Figure: 5

V & A

LOG OF TEST HOLE NO. 5 From Soil Report

Project: Reconstruction of Eubank Boulevard
Elevation - Top of Test Hole: N/A
Depth to Groundwater: Not Encountered

Project No.: 01-1-259
Date Drilled: 8/4/2001
Method: 6" HSA

Depth, feet	Blows/Foot	Sample Type	Dry Density pcf	Water Content, %	Additional Testing	Unified Classification	Material Description
							8" asphalt
1						SM	SAND, very silty, fine to coarse grained, very gravelly, medium dense, moist, brown to light brown, with thin lenses of clay
2	21	S		7.5	1		
3							
4							
5							
6	35	S		1.9			Silty, dense, medium moist, brown, no clay
7							
8							
9							
10							
11	8	S		6.5			Very silty, fine to medium grained, trace gravel, loose, moist
12							Bottom of hole at 11½'

ADDITIONAL TESTS: 1= Sieve Analysis 2= Atterberg Limits 3=Direct Shear 4=R-Value 5=Other

Figure: 6

SYSTEM 4

- NORTH OF SOUTHERN BLVD.
- BASIN 7: AREA = 0.614 ACRES

  $Q = (0.614 * 5.02) \simeq \underline{3.1\ cfs}$ ✓

- BASIN 8: A = 0.429 ACRES

  $Q = (0.429 * 5.02) \simeq \underline{2.2\ cfs}$ ✓

- BASIN 9: AREA = 1.378 ACRES

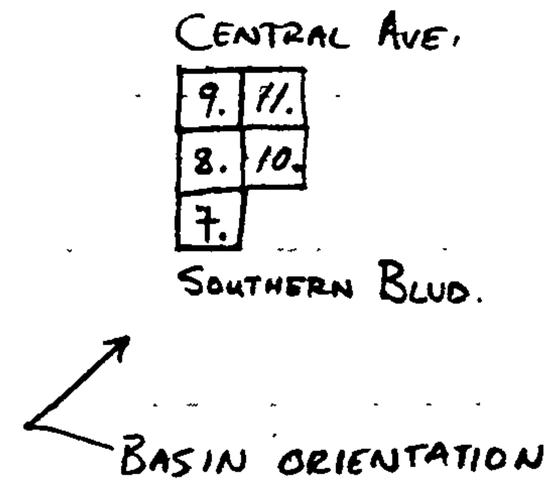
  $Q = (1.378 * 5.02) \simeq \underline{6.9\ cfs}$ ✓

- BASIN 10: AREA 2.300 ACRES

  $Q = (0.287 * 5.02) \simeq \underline{11.5\ cfs}$ ✓

- BASIN 11: AREA = 1.184 ACRES

  $Q = (1.184 * 5.02) \simeq \underline{6.0\ cfs}$ ✓



- 100-YR DESIGN STORM
- ZONE 3

- BASIN 10 AND 11 CONTRIBUTE FLOWS ENTERING CATCH BASINS AT SOUTHERN "HOME DEPOT" ENTRANCE.

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .005

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.17	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.28	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.36	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.44	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.51	0.78	0.00	0.05
0.06	0.06	0.03	0.0	0.99	0.58	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.64	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.70	1.25	0.01	0.09
0.09	0.09	0.06	0.0	1.48	0.76	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.81	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	0.86	1.72	0.01	0.12
0.12	0.12	0.11	0.1	1.97	0.92	1.88	0.01	0.13
0.13	0.13	0.13	0.1	2.14	0.97	2.03	0.01	0.14
0.14	0.14	0.16	0.1	2.66	0.93	2.54	0.01	0.15
0.15	0.15	0.18	0.2	3.18	0.92	3.06	0.01	0.16
0.16	0.16	0.22	0.2	3.70	0.93	3.57	0.01	0.17
0.17	0.17	0.25	0.2	4.22	0.95	4.08	0.01	0.18
0.18	0.18	0.30	0.3	4.73	0.98	4.59	0.01	0.19
0.19	0.19	0.35	0.3	5.25	1.01	5.10	0.02	0.21
0.20	0.20	0.40	0.4	5.77	1.04	5.61	0.02	0.22
0.21	0.21	0.46	0.5	6.29	1.08	6.12	0.02	0.23
0.22	0.22	0.52	0.6	6.81	1.12	6.64	0.02	0.24
0.23	0.23	0.59	0.7	7.33	1.15	7.15	0.02	0.25
0.24	0.24	0.67	0.8	7.85	1.19	7.66	0.02	0.26
0.25	0.25	0.74	0.9	8.37	1.23	8.17	0.02	0.27
0.26	0.26	0.83	1.1	8.89	1.27	8.68	0.03	0.29
0.27	0.27	0.92	1.2	9.41	1.31	9.19	0.03	0.30
0.28	0.28	1.01	1.4	9.93	1.35	9.70	0.03	0.31
0.29	0.29	1.11	1.5	10.45	1.39	10.21	0.03	0.32
0.30	0.30	1.22	1.7	10.97	1.43	10.73	0.03	0.33
0.31	0.31	1.33	1.9	11.48	1.47	11.24	0.03	0.34
0.32	0.32	1.44	2.2	12.00	1.50	11.75	0.04	0.36
0.33	0.33	1.56	2.4	12.52	1.54	12.26	0.04	0.37
0.34	0.34	1.69	2.7	13.04	1.58	12.77	0.04	0.38
0.35	0.35	1.82	2.9	13.56	1.62	13.28	0.04	0.39
0.36	0.36	1.95	3.2	14.08	1.66	13.79	0.04	0.40
0.37	0.37	2.09	3.5	14.60	1.69	14.31	0.04	0.41
0.38	0.38	2.24	3.9	15.12	1.73	14.82	0.05	0.43
0.39	0.39	2.39	4.2	15.64	1.77	15.33	0.05	0.44
0.40	0.40	2.54	4.6	16.16	1.80	15.84	0.05	0.45
0.41	0.41	2.71	5.0	16.68	1.84	16.35	0.05	0.46
0.42	0.42	2.87	5.4	17.20	1.87	16.86	0.05	0.47
0.43	0.43	3.04	5.8	17.72	1.91	17.37	0.06	0.49
0.44	0.44	3.22	6.3	18.23	1.95	17.89	0.06	0.50
0.45	0.45	3.40	6.7	18.75	1.98	18.40	0.06	0.51

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	7.7	19.79	2.05	19.42	0.07	0.54
0.48	0.48	3.98	8.3	20.29	2.09	19.91	0.07	0.55
0.49	0.49	4.18	8.9	20.79	2.12	20.40	0.07	0.56
0.50	0.50	4.38	9.4	21.29	2.16	20.89	0.07	0.57
0.51	0.51	4.59	10.1	21.78	2.19	21.38	0.07	0.58

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0066

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.20	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.32	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.42	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.51	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.59	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.66	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.73	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.80	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	0.87	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.93	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	0.99	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.05	1.88	0.02	0.14
0.13	0.13	0.13	0.1	2.14	1.11	2.03	0.02	0.15
0.14	0.14	0.16	0.2	2.66	1.07	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.06	3.06	0.02	0.17
0.16	0.16	0.22	0.2	3.70	1.07	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.09	4.08	0.02	0.19
0.18	0.18	0.30	0.3	4.73	1.12	4.59	0.02	0.20
0.19	0.19	0.35	0.4	5.25	1.16	5.10	0.02	0.21
0.20	0.20	0.40	0.5	5.77	1.20	5.61	0.02	0.22
0.21	0.21	0.46	0.6	6.29	1.24	6.12	0.02	0.23
0.22	0.22	0.52	0.7	6.81	1.28	6.64	0.03	0.25
0.23	0.23	0.59	0.8	7.33	1.33	7.15	0.03	0.26
0.24	0.24	0.67	0.9	7.85	1.37	7.66	0.03	0.27
0.25	0.25	0.74	1.1	8.37	1.41	8.17	0.03	0.28
0.26	0.26	0.83	1.2	8.89	1.46	8.68	0.03	0.29
0.27	0.27	0.92	1.4	9.41	1.51	9.19	0.04	0.31
0.28	0.28	1.01	1.6	9.93	1.55	9.70	0.04	0.32
0.29	0.29	1.11	1.8	10.45	1.60	10.21	0.04	0.33
0.30	0.30	1.22	2.0	10.97	1.64	10.73	0.04	0.34
0.31	0.31	1.33	2.2	11.48	1.68	11.24	0.04	0.35
0.32	0.32	1.44	2.5	12.00	1.73	11.75	0.05	0.37
0.33	0.33	1.56	2.8	12.52	1.77	12.26	0.05	0.38
0.34	0.34	1.69	3.1	13.04	1.82	12.77	0.05	0.39
0.35	0.35	1.82	3.4	13.56	1.86	13.28	0.05	0.40
0.36	0.36	1.95	3.7	14.08	1.90	13.79	0.06	0.42
0.37	0.37	2.09	4.1	14.60	1.95	14.31	0.06	0.43
0.38	0.38	2.24	4.4	15.12	1.99	14.82	0.06	0.44
0.39	0.39	2.39	4.8	15.64	2.03	15.33	0.06	0.45
0.40	0.40	2.54	5.3	16.16	2.07	15.84	0.07	0.47
0.41	0.41	2.71	5.7	16.68	2.11	16.35	0.07	0.48
0.42	0.42	2.87	6.2	17.20	2.15	16.86	0.07	0.49
0.43	0.43	3.04	6.7	17.72	2.19	17.37	0.07	0.50
0.44	0.44	3.22	7.2	18.23	2.23	17.89	0.08	0.52
0.45	0.45	3.40	7.7	18.75	2.28	18.40	0.08	0.53

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	8.9	19.79	2.35	19.42	0.09	0.56
0.48	0.48	3.98	9.5	20.29	2.40	19.91	0.09	0.57
0.49	0.49	4.18	10.2	20.79	2.44	20.40	0.09	0.58
0.50	0.50	4.38	10.9	21.29	2.48	20.89	0.10	0.60
0.51	0.51	4.59	11.6	21.78	2.52	21.38	0.10	0.61

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .007

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.21	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.33	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.43	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.52	0.63	0.00	0.04
0.05	0.05	0.02	0.0	0.82	0.60	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.68	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.76	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.83	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	0.89	1.41	0.01	0.10
0.10	0.10	0.08	0.1	1.64	0.96	1.56	0.01	0.11
0.11	0.11	0.09	0.1	1.81	1.02	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.08	1.88	0.02	0.14
0.13	0.13	0.13	0.2	2.14	1.14	2.03	0.02	0.15
0.14	0.14	0.16	0.2	2.66	1.10	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.09	3.06	0.02	0.17
0.16	0.16	0.22	0.2	3.70	1.10	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.13	4.08	0.02	0.19
0.18	0.18	0.30	0.3	4.73	1.16	4.59	0.02	0.20
0.19	0.19	0.35	0.4	5.25	1.19	5.10	0.02	0.21
0.20	0.20	0.40	0.5	5.77	1.23	5.61	0.02	0.22
0.21	0.21	0.46	0.6	6.29	1.28	6.12	0.03	0.24
0.22	0.22	0.52	0.7	6.81	1.32	6.64	0.03	0.25
0.23	0.23	0.59	0.8	7.33	1.37	7.15	0.03	0.26
0.24	0.24	0.67	0.9	7.85	1.41	7.66	0.03	0.27
0.25	0.25	0.74	1.1	8.37	1.46	8.17	0.03	0.28
0.26	0.26	0.83	1.2	8.89	1.50	8.68	0.04	0.30
0.27	0.27	0.92	1.4	9.41	1.55	9.19	0.04	0.31
0.28	0.28	1.01	1.6	9.93	1.60	9.70	0.04	0.32
0.29	0.29	1.11	1.8	10.45	1.64	10.21	0.04	0.33
0.30	0.30	1.22	2.1	10.97	1.69	10.73	0.04	0.34
0.31	0.31	1.33	2.3	11.48	1.73	11.24	0.05	0.36
0.32	0.32	1.44	2.6	12.00	1.78	11.75	0.05	0.37
0.33	0.33	1.56	2.9	12.52	1.83	12.26	0.05	0.38
0.34	0.34	1.69	3.2	13.04	1.87	12.77	0.05	0.39
0.35	0.35	1.82	3.5	13.56	1.91	13.28	0.06	0.41
0.36	0.36	1.95	3.8	14.08	1.96	13.79	0.06	0.42
0.37	0.37	2.09	4.2	14.60	2.00	14.31	0.06	0.43
0.38	0.38	2.24	4.6	15.12	2.05	14.82	0.07	0.45
0.39	0.39	2.39	5.0	15.64	2.09	15.33	0.07	0.46
0.40	0.40	2.54	5.4	16.16	2.13	15.84	0.07	0.47
0.41	0.41	2.71	5.9	16.68	2.18	16.35	0.07	0.48
0.42	0.42	2.87	6.4	17.20	2.22	16.86	0.08	0.50
0.43	0.43	3.04	6.9	17.72	2.26	17.37	0.08	0.51
0.44	0.44	3.22	7.4	18.23	2.30	17.89	0.08	0.52
0.45	0.45	3.40	8.0	18.75	2.34	18.40	0.09	0.54

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	9.2	19.79	2.42	19.42	0.09	0.56
0.48	0.48	3.98	9.8	20.29	2.47	19.91	0.09	0.57
0.49	0.49	4.18	10.5	20.79	2.51	20.40	0.10	0.59
0.50	0.50	4.38	11.2	21.29	2.55	20.89	0.10	0.60
0.51	0.51	4.59	11.9	21.78	2.59	21.38	0.10	0.61

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0091

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.24	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.37	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.49	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.59	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.69	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.78	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.86	1.09	0.01	0.08
0.08	0.08	0.05	0.0	1.32	0.94	1.25	0.01	0.09
0.09	0.09	0.06	0.1	1.48	1.02	1.41	0.02	0.11
0.10	0.10	0.08	0.1	1.64	1.09	1.56	0.02	0.12
0.11	0.11	0.09	0.1	1.81	1.17	1.72	0.02	0.13
0.12	0.12	0.11	0.1	1.97	1.24	1.88	0.02	0.14
0.13	0.13	0.13	0.2	2.14	1.30	2.03	0.03	0.16
0.14	0.14	0.16	0.2	2.66	1.25	2.54	0.02	0.16
0.15	0.15	0.18	0.2	3.18	1.24	3.06	0.02	0.17
0.16	0.16	0.22	0.3	3.70	1.26	3.57	0.02	0.18
0.17	0.17	0.25	0.3	4.22	1.28	4.08	0.03	0.20
0.18	0.18	0.30	0.4	4.73	1.32	4.59	0.03	0.21
0.19	0.19	0.35	0.5	5.25	1.36	5.10	0.03	0.22
0.20	0.20	0.40	0.6	5.77	1.41	5.61	0.03	0.23
0.21	0.21	0.46	0.7	6.29	1.45	6.12	0.03	0.24
0.22	0.22	0.52	0.8	6.81	1.50	6.64	0.04	0.26
0.23	0.23	0.59	0.9	7.33	1.56	7.15	0.04	0.27
0.24	0.24	0.67	1.1	7.85	1.61	7.66	0.04	0.28
0.25	0.25	0.74	1.2	8.37	1.66	8.17	0.04	0.29
0.26	0.26	0.83	1.4	8.89	1.71	8.68	0.05	0.31
0.27	0.27	0.92	1.6	9.41	1.77	9.19	0.05	0.32
0.28	0.28	1.01	1.8	9.93	1.82	9.70	0.05	0.33
0.29	0.29	1.11	2.1	10.45	1.87	10.21	0.05	0.34
0.30	0.30	1.22	2.3	10.97	1.93	10.73	0.06	0.36
0.31	0.31	1.33	2.6	11.48	1.98	11.24	0.06	0.37
0.32	0.32	1.44	2.9	12.00	2.03	11.75	0.06	0.38
0.33	0.33	1.56	3.2	12.52	2.08	12.26	0.07	0.40
0.34	0.34	1.69	3.6	13.04	2.13	12.77	0.07	0.41
0.35	0.35	1.82	4.0	13.56	2.18	13.28	0.07	0.42
0.36	0.36	1.95	4.4	14.08	2.23	13.79	0.08	0.44
0.37	0.37	2.09	4.8	14.60	2.28	14.31	0.08	0.45
0.38	0.38	2.24	5.2	15.12	2.33	14.82	0.08	0.46
0.39	0.39	2.39	5.7	15.64	2.38	15.33	0.09	0.48
0.40	0.40	2.54	6.2	16.16	2.43	15.84	0.09	0.49
0.41	0.41	2.71	6.7	16.68	2.48	16.35	0.10	0.51
0.42	0.42	2.87	7.3	17.20	2.53	16.86	0.10	0.52
0.43	0.43	3.04	7.8	17.72	2.58	17.37	0.10	0.53
0.44	0.44	3.22	8.4	18.23	2.62	17.89	0.11	0.55
0.45	0.45	3.40	9.1	18.75	2.67	18.40	0.11	0.56

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	10.4	19.79	2.76	19.42	0.12	0.59
0.48	0.48	3.98	11.2	20.29	2.81	19.91	0.12	0.60
0.49	0.49	4.18	11.9	20.79	2.86	20.40	0.13	0.62
0.50	0.50	4.38	12.7	21.29	2.91	20.89	0.13	0.63
0.51	0.51	4.59	13.6	21.78	2.95	21.38	0.14	0.65

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0121

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.27	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.43	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.57	0.47	0.00	0.03
0.04	0.04	0.01	0.0	0.66	0.68	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.79	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	0.90	0.94	0.01	0.07
0.07	0.07	0.04	0.0	1.15	0.99	1.09	0.02	0.09
0.08	0.08	0.05	0.1	1.32	1.09	1.25	0.02	0.10
0.09	0.09	0.06	0.1	1.48	1.18	1.41	0.02	0.11
0.10	0.10	0.08	0.1	1.64	1.26	1.56	0.02	0.12
0.11	0.11	0.09	0.1	1.81	1.34	1.72	0.03	0.14
0.12	0.12	0.11	0.2	1.97	1.42	1.88	0.03	0.15
0.13	0.13	0.13	0.2	2.14	1.50	2.03	0.04	0.17
0.14	0.14	0.16	0.2	2.66	1.45	2.54	0.03	0.17
0.15	0.15	0.18	0.3	3.18	1.43	3.06	0.03	0.18
0.16	0.16	0.22	0.3	3.70	1.45	3.57	0.03	0.19
0.17	0.17	0.25	0.4	4.22	1.48	4.08	0.03	0.20
0.18	0.18	0.30	0.5	4.73	1.52	4.59	0.04	0.22
0.19	0.19	0.35	0.5	5.25	1.57	5.10	0.04	0.23
0.20	0.20	0.40	0.6	5.77	1.62	5.61	0.04	0.24
0.21	0.21	0.46	0.8	6.29	1.68	6.12	0.04	0.25
0.22	0.22	0.52	0.9	6.81	1.74	6.64	0.05	0.27
0.23	0.23	0.59	1.1	7.33	1.79	7.15	0.05	0.28
0.24	0.24	0.67	1.2	7.85	1.85	7.66	0.05	0.29
0.25	0.25	0.74	1.4	8.37	1.92	8.17	0.06	0.31
0.26	0.26	0.83	1.6	8.89	1.98	8.68	0.06	0.32
0.27	0.27	0.92	1.9	9.41	2.04	9.19	0.06	0.33
0.28	0.28	1.01	2.1	9.93	2.10	9.70	0.07	0.35
0.29	0.29	1.11	2.4	10.45	2.16	10.21	0.07	0.36
0.30	0.30	1.22	2.7	10.97	2.22	10.73	0.08	0.38
0.31	0.31	1.33	3.0	11.48	2.28	11.24	0.08	0.39
0.32	0.32	1.44	3.4	12.00	2.34	11.75	0.09	0.41
0.33	0.33	1.56	3.7	12.52	2.40	12.26	0.09	0.42
0.34	0.34	1.69	4.1	13.04	2.46	12.77	0.09	0.43
0.35	0.35	1.82	4.6	13.56	2.52	13.28	0.10	0.45
0.36	0.36	1.95	5.0	14.08	2.58	13.79	0.10	0.46
0.37	0.37	2.09	5.5	14.60	2.63	14.31	0.11	0.48
0.38	0.38	2.24	6.0	15.12	2.69	14.82	0.11	0.49
0.39	0.39	2.39	6.6	15.64	2.75	15.33	0.12	0.51
0.40	0.40	2.54	7.1	16.16	2.80	15.84	0.12	0.52
0.41	0.41	2.71	7.7	16.68	2.86	16.35	0.13	0.54
0.42	0.42	2.87	8.4	17.20	2.92	16.86	0.13	0.55
0.43	0.43	3.04	9.0	17.72	2.97	17.37	0.14	0.57
0.44	0.44	3.22	9.7	18.23	3.03	17.89	0.14	0.58
0.45	0.45	3.40	10.5	18.75	3.08	18.40	0.15	0.60

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	12.0	19.79	3.19	19.42	0.16	0.63
0.48	0.48	3.98	12.9	20.29	3.24	19.91	0.16	0.64
0.49	0.49	4.18	13.8	20.79	3.30	20.40	0.17	0.66
0.50	0.50	4.38	14.7	21.29	3.35	20.89	0.17	0.67
0.51	0.51	4.59	15.7	21.78	3.41	21.38	0.18	0.69

Typical Street Section Flow Capacity Calculations Based on Manning's Equation

MANNING'S N= .017 SLOPE= .0157

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.67	3	2.17	0.13	5	31.17	0.71
2	0.17	0.00	4	19.47	0.47	6	0.00	0.00

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.01	0.01	0.00	0.0	0.16	0.31	0.16	0.00	0.01
0.02	0.02	0.00	0.0	0.33	0.49	0.31	0.00	0.02
0.03	0.03	0.01	0.0	0.49	0.64	0.47	0.01	0.04
0.04	0.04	0.01	0.0	0.66	0.78	0.63	0.01	0.05
0.05	0.05	0.02	0.0	0.82	0.91	0.78	0.01	0.06
0.06	0.06	0.03	0.0	0.99	1.02	0.94	0.02	0.08
0.07	0.07	0.04	0.0	1.15	1.13	1.09	0.02	0.09
0.08	0.08	0.05	0.1	1.32	1.24	1.25	0.02	0.10
0.09	0.09	0.06	0.1	1.48	1.34	1.41	0.03	0.12
0.10	0.10	0.08	0.1	1.64	1.44	1.56	0.03	0.13
0.11	0.11	0.09	0.1	1.81	1.53	1.72	0.04	0.15
0.12	0.12	0.11	0.2	1.97	1.62	1.88	0.04	0.16
0.13	0.13	0.13	0.2	2.14	1.71	2.03	0.05	0.18
0.14	0.14	0.16	0.3	2.66	1.65	2.54	0.04	0.18
0.15	0.15	0.18	0.3	3.18	1.63	3.06	0.04	0.19
0.16	0.16	0.22	0.4	3.70	1.65	3.57	0.04	0.20
0.17	0.17	0.25	0.4	4.22	1.69	4.08	0.04	0.21
0.18	0.18	0.30	0.5	4.73	1.73	4.59	0.05	0.23
0.19	0.19	0.35	0.6	5.25	1.79	5.10	0.05	0.24
0.20	0.20	0.40	0.7	5.77	1.85	5.61	0.05	0.25
0.21	0.21	0.46	0.9	6.29	1.91	6.12	0.06	0.27
0.22	0.22	0.52	1.0	6.81	1.98	6.64	0.06	0.28
0.23	0.23	0.59	1.2	7.33	2.04	7.15	0.06	0.29
0.24	0.24	0.67	1.4	7.85	2.11	7.66	0.07	0.31
0.25	0.25	0.74	1.6	8.37	2.18	8.17	0.07	0.32
0.26	0.26	0.83	1.9	8.89	2.25	8.68	0.08	0.34
0.27	0.27	0.92	2.1	9.41	2.32	9.19	0.08	0.35
0.28	0.28	1.01	2.4	9.93	2.39	9.70	0.09	0.37
0.29	0.29	1.11	2.7	10.45	2.46	10.21	0.09	0.38
0.30	0.30	1.22	3.1	10.97	2.53	10.73	0.10	0.40
0.31	0.31	1.33	3.4	11.48	2.60	11.24	0.10	0.41
0.32	0.32	1.44	3.8	12.00	2.67	11.75	0.11	0.43
0.33	0.33	1.56	4.3	12.52	2.73	12.26	0.12	0.45
0.34	0.34	1.69	4.7	13.04	2.80	12.77	0.12	0.46
0.35	0.35	1.82	5.2	13.56	2.87	13.28	0.13	0.48
0.36	0.36	1.95	5.7	14.08	2.93	13.79	0.13	0.49
0.37	0.37	2.09	6.3	14.60	3.00	14.31	0.14	0.51
0.38	0.38	2.24	6.9	15.12	3.07	14.82	0.15	0.53
0.39	0.39	2.39	7.5	15.64	3.13	15.33	0.15	0.54
0.40	0.40	2.54	8.1	16.16	3.19	15.84	0.16	0.56
0.41	0.41	2.71	8.8	16.68	3.26	16.35	0.16	0.57
0.42	0.42	2.87	9.5	17.20	3.32	16.86	0.17	0.59
0.43	0.43	3.04	10.3	17.72	3.38	17.37	0.18	0.61
0.44	0.44	3.22	11.1	18.23	3.45	17.89	0.18	0.62
0.45	0.45	3.40	11.9	18.75	3.51	18.40	0.19	0.64

WSEL (FT)	DEPTH INC (FT)	FLOW AREA SQ.FT.	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOPWID (FT)	VEL HEAD (FT)	ENERGY HEAD (FT)
0.47	0.47	3.78	13.7	19.79	3.63	19.42	0.20	0.67
0.48	0.48	3.98	14.7	20.29	3.69	19.91	0.21	0.69
0.49	0.49	4.18	15.7	20.79	3.76	20.40	0.22	0.71
0.50	0.50	4.38	16.7	21.29	3.82	20.89	0.23	0.73
0.51	0.51	4.59	17.8	21.78	3.88	21.38	0.23	0.74

Single A inlet, in sump condition with curb openings on both sides:
Open Area (for orifice calc in sq. ft.): 3.9314236
Length of Weir (feet): 11.354167

Head (ft)	Head (in)	Weir Q	Orifice Q	Control Q
0.05	0.6	0.34	4.23	0.34
0.1	1.2	0.96	5.99	0.96
0.15	1.8	1.77	7.33	1.77
0.2	2.4	2.72	8.47	2.72
0.25	3	3.80	9.46	3.80
0.3	3.6	5.00	10.37	5.00
0.35	4.2	6.30	11.20	6.30
0.4	4.8	7.70	11.97	7.70
0.45	5.4	9.19	12.70	9.19
0.5	6	10.76	13.39	10.76
0.55	6.6	12.41	14.04	12.41
0.6	7.2	14.14	14.66	14.14
0.65	7.8	15.95	15.26	15.26
0.67	8.04	16.69	15.49	15.49
0.7	8.4	17.82	15.84	15.84
0.75	9	19.76	16.39	16.39
0.8	9.6	21.77	16.93	16.93
0.85	10.2	23.85	17.45	17.45
0.9	10.8	25.98	17.96	17.96
0.95	11.4	28.18	18.45	18.45
1	12	30.43	18.93	18.93
1.05	12.6	32.74	19.40	19.40
1.1	13.2	35.11	19.85	19.85
1.15	13.8	37.53	20.30	20.30
1.2	14.4	40.00	20.74	20.74

Calculation of open area:

	(in^2)	(ft^2)
Total Grate Area	1000	6.9444444
Cross Bar Area	-366	-2.541667
Supports (ends)	-115.625	-0.802951
Areas Counted Twice	47.75	0.3315972
	566.125	3.9314236

Calculation of Length of Weir:

	(in)	(ft)
Total Perimeter of Grate	90	7.5
Short Cross Bars	-3.5	-0.291667
End Supports	-9.25	-0.770833
Bearing Bars	-13	-1.083333
Curb Openings	72	6
	136.25	11.354167

Single D inlet, in sump condition:

Open Area (for orifice calc in sq. ft.): 3.93

Length of Weir (feet): 8.40

Head (ft)	Head (in)	Weir Q	Orifice Q	Control Q
0.05	0.6	0.25	4.23	0.25
0.1	1.2	0.71	5.99	0.71
0.15	1.8	1.31	7.33	1.31
0.2	2.4	2.01	8.47	2.01
0.25	3	2.81	9.46	2.81
0.3	3.6	3.70	10.37	3.70
0.35	4.2	4.66	11.20	4.66
0.4	4.8	5.69	11.97	5.69
0.45	5.4	6.79	12.70	6.79
0.5	6	7.96	13.39	7.96
0.55	6.6	9.18	14.04	9.18
0.6	7.2	10.46	14.66	10.46
0.65	7.8	11.79	15.26	11.79
0.67	8.04	12.34	15.49	12.34
0.7	8.4	13.18	15.84	13.18
0.75	9	14.61	16.39	14.61
0.8	9.6	16.10	16.93	16.10
0.85	10.2	17.63	17.45	17.45
0.9	10.8	19.21	17.96	17.96
0.95	11.4	20.83	18.45	18.45
1	12	22.50	18.93	18.93
1.05	12.6	24.21	19.40	19.40
1.1	13.2	25.96	19.85	19.85
1.15	13.8	27.75	20.30	20.30
1.2	14.4	29.58	20.74	20.74
1.25	15	31.45	21.16	21.16
1.3	15.6	33.35	21.58	21.58

Calculation of open area:

	(in^2)	(ft^2)
Total Grate Area	1000	6.944
Cross Bar Area	-366	-2.542
Supports (ends)	-115.625	-0.803
Areas Counted Twice	47.75	0.332
	566.125	3.931

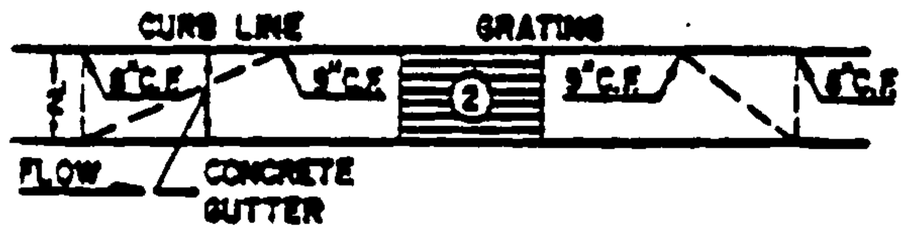
Calculation of Length of Weir (four sides open):

	(in)	(ft)
Total Perimeter of Grate	130	10.833
Short Cross Bars	-7	-0.583
End Supports	-9.25	-0.771
Bearing Bars	-13	-1.083
	100.75	8.396
		0.775

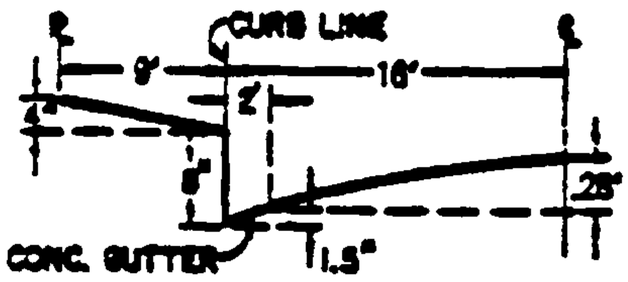
Calculation of Length of Weir (three sides open):

	(in)	(ft)
Total Perimeter of Grate	90	7.500
Short Cross Bars	-3.5	-0.292
End Supports	-9.25	-0.771
Bearing Bars	-13	-1.083
	64.25	5.354
		0.714

GRATING CAPACITIES FOR TYPE 'A', 'C' and 'D'



GRATING & GUTTER PLAN



TYPICAL HALF STREET SECTION
(ABOVE BASIN)

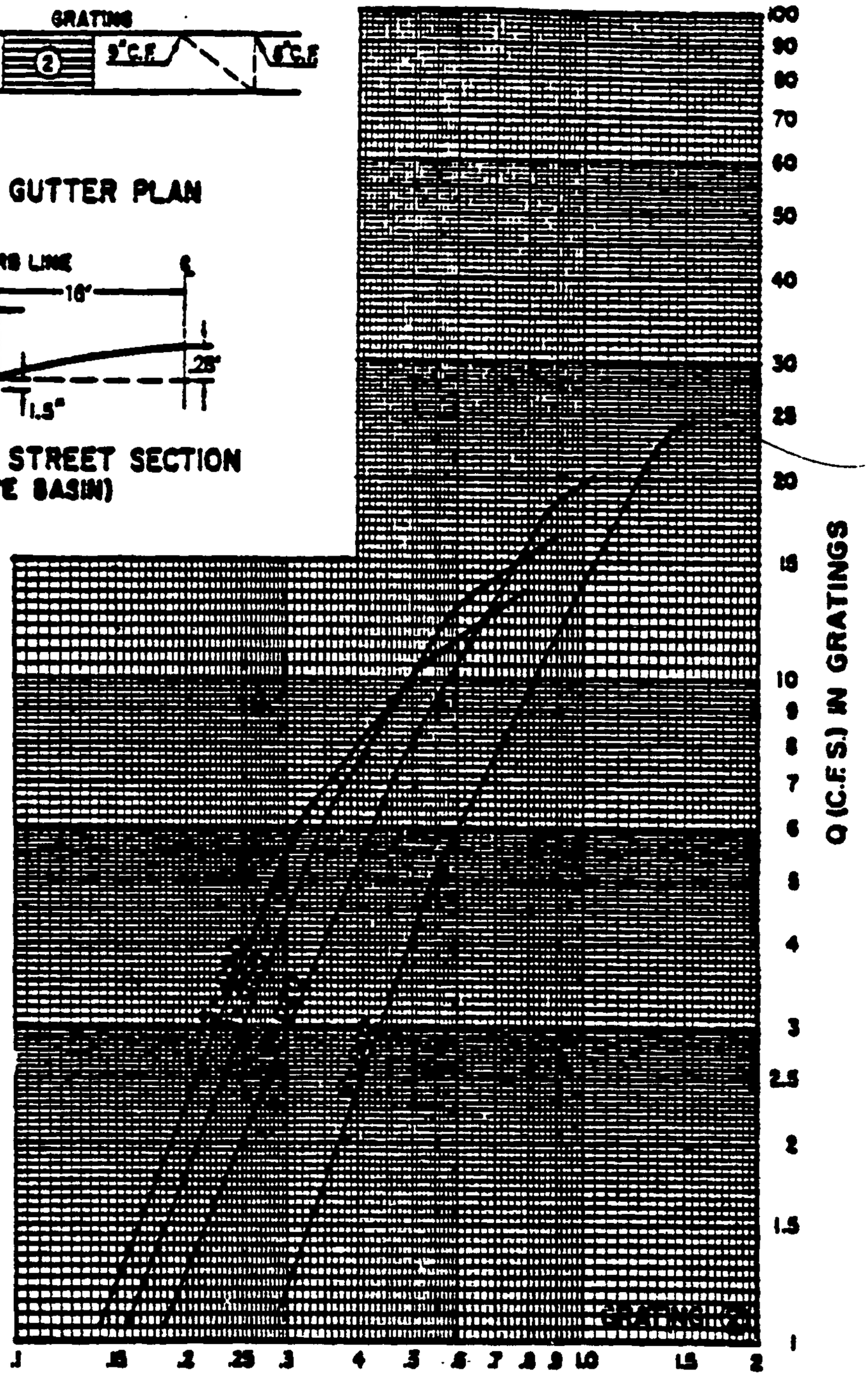


PLATE 22.3 D-5

APPENDIX D

Hydraulic Grade Line Profiles

APPENDIX E

Eubank Boulevard. 54-inch Storm Drain As-Builts

POT HOLE DATA					
PH#	UTIL	NORTHING	EASTING	ELEV	DEPTH
17	GAS	1475906 51	1556309 45	5468.69	32"
26	WATER	1475688 75	1556385 16	5467 98	67"

⟨1⟩ UTILITY CROSSING TABLE

CROSSING	SD	GAS	SAS	WL	CLEARANCE
1	TP=64 55'	BP=65 87'	N/A	N/A	1 32'
2	BP=59 57'	N/A	TP=57 65'	N/A	1.92'
3	TP=61 41'	N/A	N/A	BP=62.36'	0 95'
4	TP=66 03'	BP=67 01'	N/A	N/A	0. 98'
5	BP=64 84'	N/A	TP=62 14'	N/A	2 70'
6 *	BP=64 89'	N/A	N/A	TP=64 48'	0 41'
7	TP=67 11'	BP=67 88'	N/A	N/A	0 77

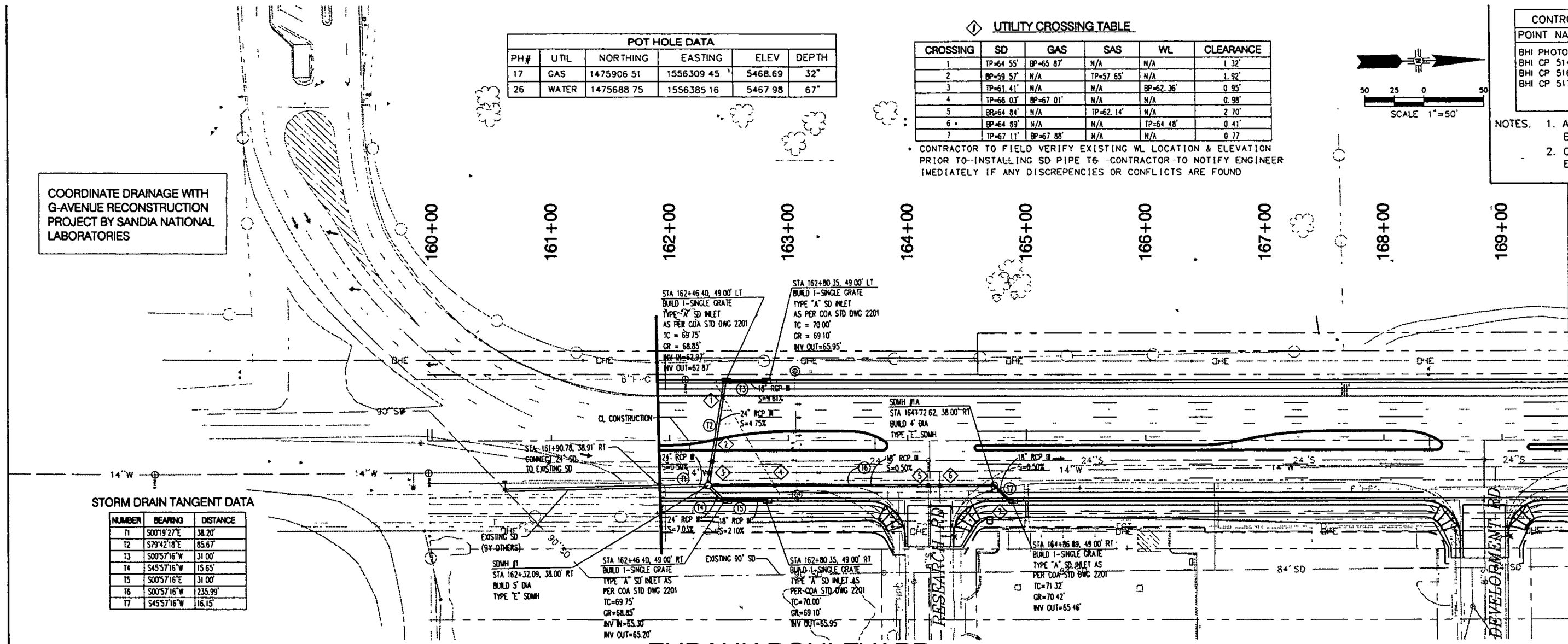
* CONTRACTOR TO FIELD VERIFY EXISTING WL LOCATION & ELEVATION PRIOR TO INSTALLING SD PIPE TO CONTRACTOR TO NOTIFY ENGINEER IMEDIATELY IF ANY DISCREPENCIES OR CONFLICTS ARE FOUND

CONTROL POINT DATA TABLE (GROUND COORDINATES)			
POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956 90	1555960 12	5463 22
BHI CP 516	1478043.06	1555862 87	5461 77
BHI CP 517	1477917 89	1557166 71	5480 08

NOTES.
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM

50 25 0 50
SCALE 1"=50'

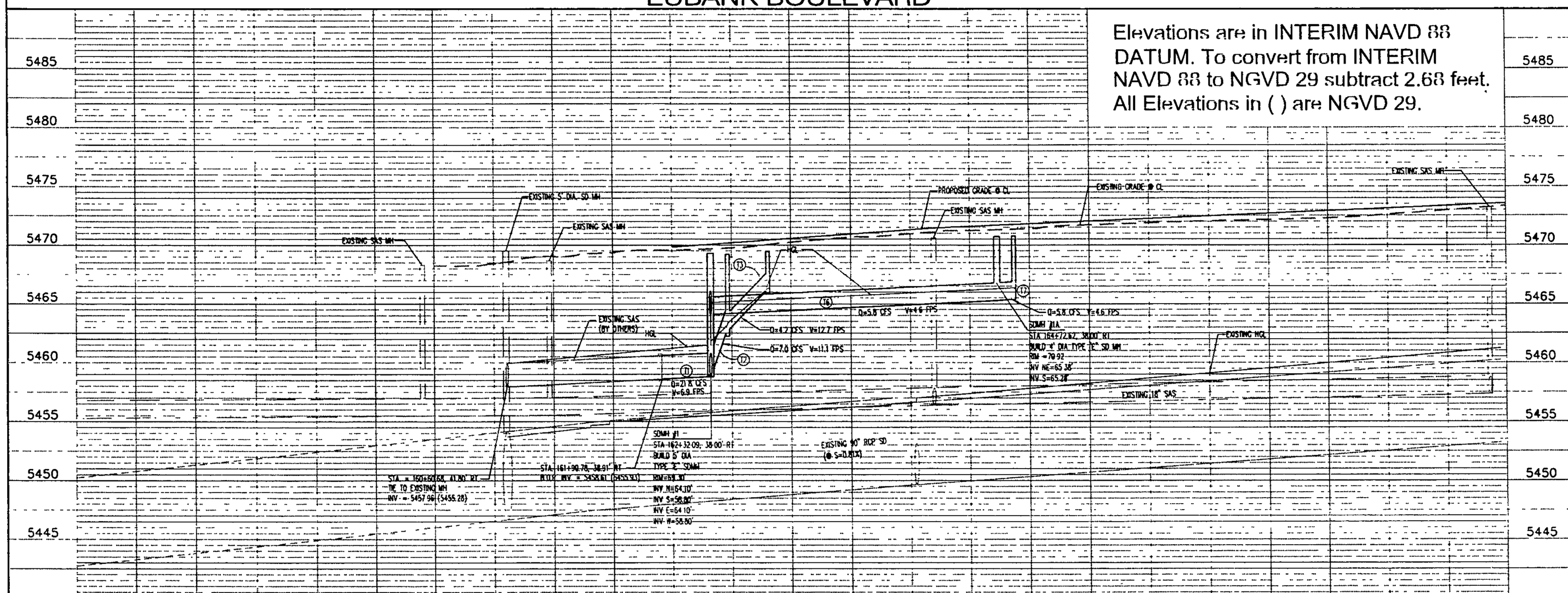
COORDINATE DRAINAGE WITH G-AVENUE RECONSTRUCTION PROJECT BY SANDIA NATIONAL LABORATORIES



STORM DRAIN TANGENT DATA

NUMBER	BEARING	DISTANCE
11	S00°19'27"E	38.20'
12	S79°42'18"E	85.67'
13	S00°57'16"W	31 00'
14	S45°57'16"W	15 65'
15	S00°57'16"E	31 00'
16	S00°57'16"W	235.99'
17	S45°57'16"W	16.15'

Elevations are in INTERIM NAVD 88 DATUM. To convert from INTERIM NAVD 88 to NGVD 29 subtract 2.68 feet. All Elevations in ( ) are NGVD 29.



SCALE: 1" = 50' (HORIZ)
1" = 5' (VERT)

P \00268\Hydro\Design\PNP\00268pnp01 dwg
Jun 19, 2002 - 4 22pm

BHI JOB NO 00268

MARCH 2002

AS-BUILT INFORMATION
CONTRACTOR — DATE
WORK STAKED BY — DATE
INSPECTORS ACCEPTANCE BY — DATE
FABRICATION BY — DATE
CORRECTED BY — DATE
MICRO-FILM INFORMATION
RECORDED BY — DATE
NO — DATE

BENCH MARKS
ACS Aluminum Cap stamped "6-L21, 1986/92"
Intersection of Central Ave & Elizabeth St Geographic Position (NAD 1927), in feet N M State Plane Coordinates (Central Zone) X=417,975 01, Y=1,481,220 09 (Grid) Elevation=5500 67 NGVD29, in feet Interim NAD 83, in feet [X= 1,558,774 59, Y= 1,481,809 99] (Ground)
Ground-to-grid factor 0 99964527
Transformed to Interim NAVD 88 by Corpscon V5 11 08
[Elevation= 5503 35 Interim NAVD88, in feet]

SURVEY INFORMATION
FIELD NOTES
NO. | BY | DATE

ENGINEER'S SEAL

No	Date	By	REMARKS
			REVISIONS

DESIGN
Designed By M P C — DATE 04/02
Drawn By S F G — DATE 04/02
Checked By M P C /C W H — DATE 04/02

Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

EUBANK BOULEVARD IMPROVEMENTS
STORM DRAIN
STA. 160+00 TO STA. 169+00

Design Review Committee | City Engineer Approval | Last Design Update | Mo /Day/ Yr | Mo /Day/ Yr

City Project No	Zone Map No	Sheet	Of
6061.01	L-21&M-21	35	114

APPENDIX F

Correspondence

TELEPHONE CONFERENCE NOTES

JOB NO. 00268

PROJECT TITLE Eubank

DATE **9/24/01**

TIME 11:40

CONVERSATION WITH Brad Bingham **OF** City of Albuquerque

AND John Gurule **OF** BHI

SUBJECT **Size of trunk line in area north of Southern Ave.**

CONTENT

I asked Brad if it was acceptable to use an 18 inch diameter pipe for the trunk line in the area north of Southern Avenue instead of the standard 24 inch pipe. His response was, "since the trunk line would be connecting to an existing 18-inch pipe in the Acoma area, it would not be beneficial to the City, to place a 24-inch pipe upstream and then narrow it down to 18 inches at the connection."

John Gurule

UTILITY CROSSING TABLE

CROSSING	SD	GAS	SAS	WL	COMM	CLEARANCE
1	BP=69.83'	N/A	TP=67.26'	N/A	TP=69.43	2.57' / 0.40'
2	BP=69.85'	N/A	N/A	TP=68.78'	N/A	1.07'
3	TP=71.70'	N/A	N/A	N/A	BP=72.13'	0.43'
4	BP=62.24'	N/A	TP=62.04'	N/A	N/A	0.20'
5	TP=64.73'	N/A	N/A	N/A	BP=68.07' / BP=67.82'	3.34' / 3.09'
6	TP=64.77'	N/A	N/A	N/A	BP=69.69'	4.92'

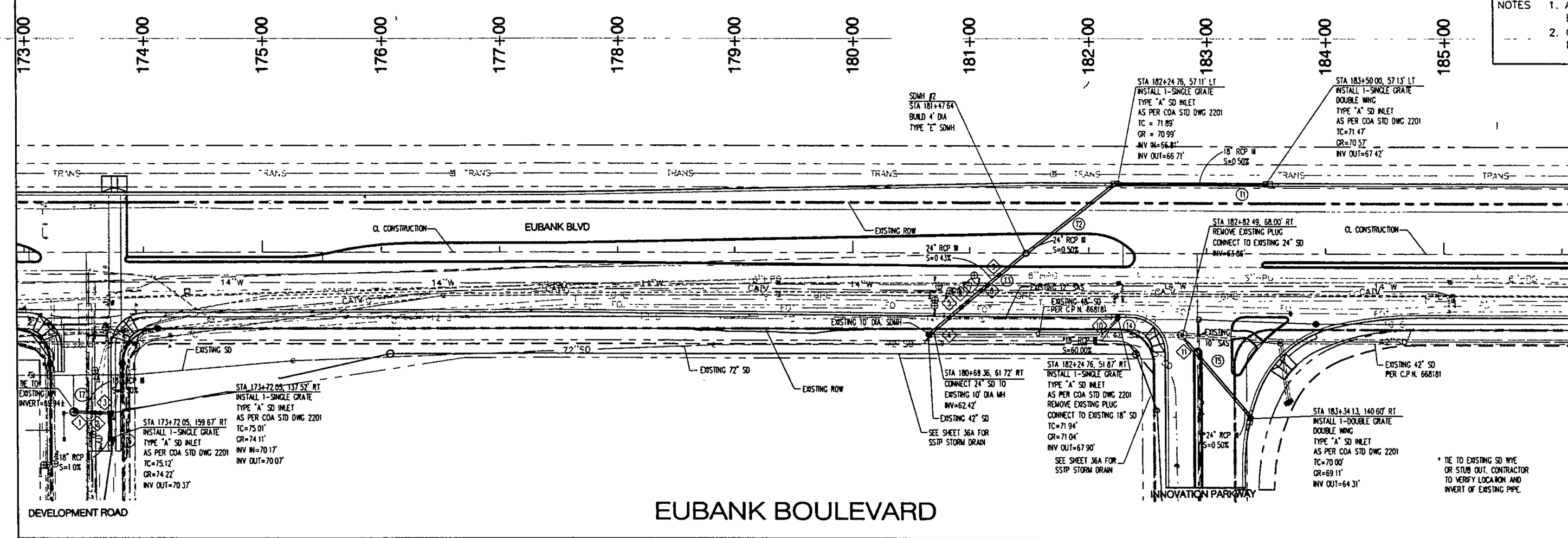
UTILITY CROSSING TABLE

CROSSING	SD	GAS	SAS	WL	COMM	CLEARANCE
7			CONFLICT WITH CATV, MUST ADJUST CATV			
8	TP=64.88'	N/A	N/A	BP=65.09'	N/A	0.21'
9	TP=64.95'	BP=67.51'	N/A	N/A	N/A	2.56'
10	BP=67.73'	N/A	TP=62.53'	N/A	N/A	5.20'
11	BP=63.75'	N/A	TP=63.25'	N/A	N/A	0.50'

CONTROL POINT DATA TABLE (GROUND COORDINATES)

POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956.90	1555960.12	5463.22
BHI CP 516	1478043.06	1555862.87	5461.77
BHI CP 517	1477917.89	1557166.71	5480.08

NOTES
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM



STORM DRAIN TANGENT DATA

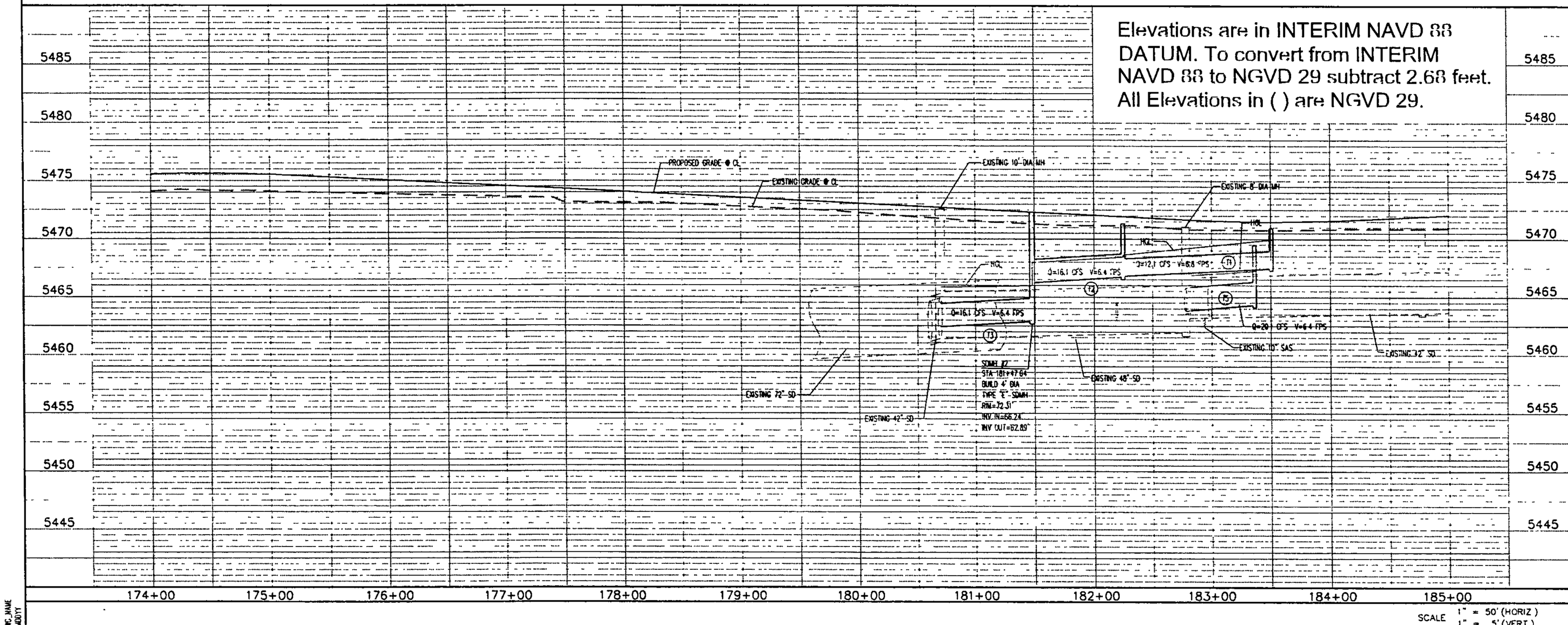
NUMBER	BEARING	DISTANCE
T1	S00°56'59"W	122.32'
T2	S37°15'43"E	94.23'
T3	S39°04'49"E	108.28'
T4	S44°02'46"E	8.00'
T5	S54°02'58"W	89.53'
T6	N88°55'26"W	19.33'
T7	S03°09'53"W	25.76'

POT HOLE DATA

PH #	UTIL	NORTHING	EASTING	ELEV	DEPTH
5	GAS	1477795.48	1556412.00	5471.29	34"
6	CATV	1477774.30	1556424.92	5471.60	81"
7	WATER	1477764.59	1556420.66	5471.59	58"
8	F.O.	1477760.41	1556437.55	5472.29	28"
9	COMM	1477761.00	1556432.81	5472.24	24"

POT HOLE DATA

PH #	UTIL	NORTHING	EASTING	ELEV	DEPTH
10	GAS	1477014.20	1556560.24	5475.73	36"
11	ELEC	1477007.06	1556561.67	5475.91	38"
12	F.O.	1477004.45	1556561.57	5475.92	18"
13	WATER	1477048.26	1556558.38	5474.96	CLEAR TO 96"
27	WATER	1477049.16	1556481.43	5474.67	72"
28	COMM	1477051.18	1556540.07	5475.33	33"



DWG_NAME
MMDDYY

P:\00268\Hydro\Design\PNP\00268pnp02.dwg
Jun 19, 2002 - 4:23pm

BHI JOB NO 00268

MARCH 2002

AS-BUILT INFORMATION
CONTRACTOR DATE
WORK STAKED BY DATE
INSPECTOR'S ACCEPTANCE BY DATE
FIELD VERIFICATION BY DATE
DRAWINGS CORRECTION BY DATE

BENCH MARKS
ACS Aluminum Cap stamped "6-L21, 1986/92"
Intersection of Central Ave. & Elizabeth St. Geographic Position (NAD 1927), in feet N.M. State Plane Coordinates (Central Zone) X=417,975.01, Y=1,481,220.09 (Grid) Elevation=5500.67 NGVD29, in feet Interim NAD 83 in feet [X= 1,558,774.59, Y= 1,481,809.99] (Ground)
Ground-to-grid factor 0.99964527
Transformed to Interim NAVD 88 by Carpscon V5.11.08 [Elevation= 5503.35 Interim NAVD88, in feet]

MICRO-FILM INFORMATION
RECORDED BY DATE
NO

SURVEY INFORMATION
FIELD NOTES

NO	BY	DATE

ENGINEER'S SEAL

No.	Date	REMARKS	By
		REVISIONS	
		DESIGN	

Designed By M.P.C. DATE 04/02
Drawn By MTD DATE 04/02
Checked By M.P.C./C.W.H. DATE 04/02

Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

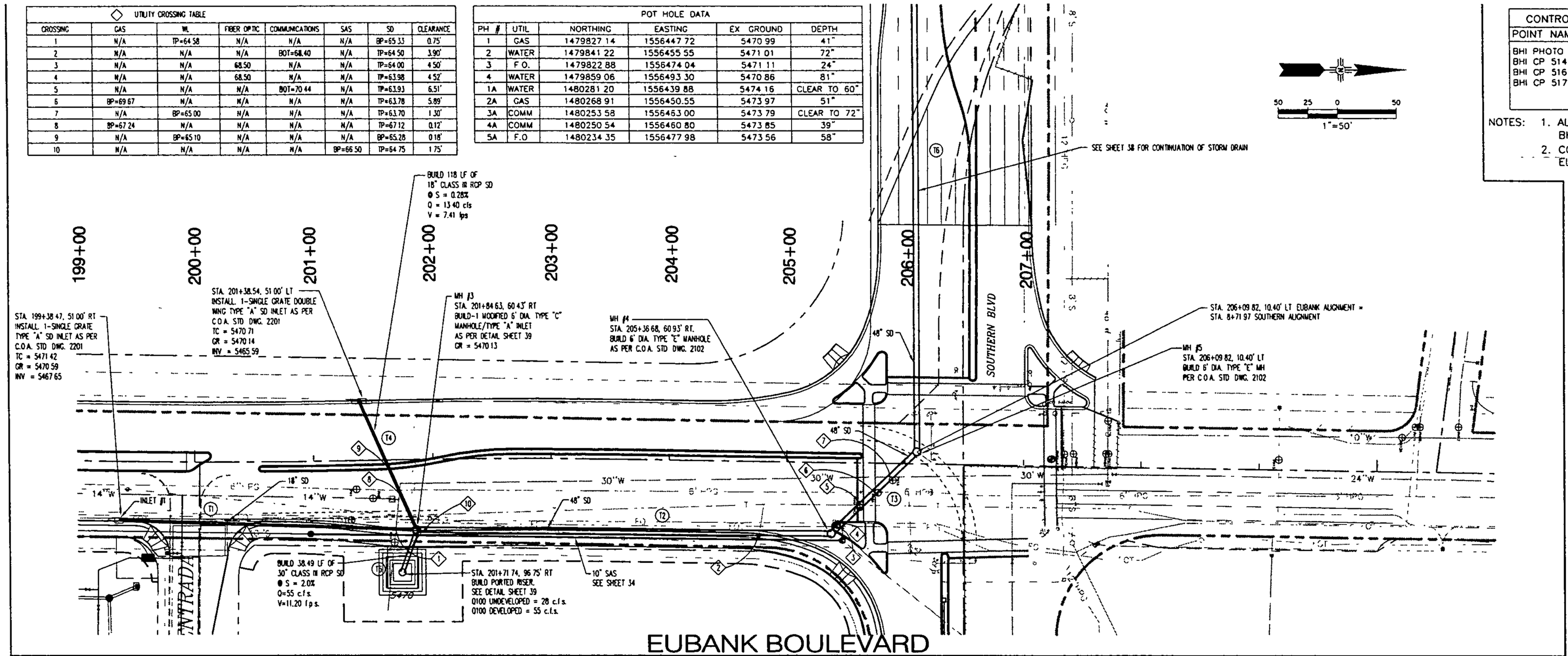
EUBANK BOULEVARD IMPROVEMENTS
STORM DRAIN
STA. 174+00 TO STA. 185+00

Design Review Committee	City Engineer Approval	Last Design Update	Mo./Day/Yr	Mo./Day/Yr

City Project No	Zone Map No.	Sheet	Of
6061.01	L-21&M-21	36	114

◇ UTILITY CROSSING TABLE							
CROSSING	GAS	WL	FIBER OPTIC	COMMUNICATIONS	SAS	SD	CLEARANCE
1	N/A	TP=64.58	N/A	N/A	N/A	BP=65.33	0.75'
2	N/A	N/A	N/A	BOT=68.40	N/A	TP=64.50	3.90'
3	N/A	N/A	68.50	N/A	N/A	TP=64.00	4.50'
4	N/A	N/A	68.50	N/A	N/A	TP=63.98	4.52'
5	N/A	N/A	N/A	BOT=70.44	N/A	TP=63.93	6.51'
6	BP=69.67	N/A	N/A	N/A	N/A	TP=63.78	5.89'
7	N/A	BP=65.00	N/A	N/A	N/A	TP=63.70	1.30'
8	BP=67.24	N/A	N/A	N/A	N/A	TP=67.12	0.12'
9	N/A	BP=65.10	N/A	N/A	N/A	BP=65.28	0.18'
10	N/A	N/A	N/A	N/A	BP=66.50	TP=64.75	1.75'

POT HOLE DATA					
PH #	UTIL	NORTHING	EASTING	EX GROUND	DEPTH
1	GAS	1479827.14	1556447.72	5470.99	41"
2	WATER	1479841.22	1556455.55	5471.01	72"
3	F.O.	1479822.88	1556474.04	5471.11	24"
4	WATER	1479859.06	1556493.30	5470.86	81"
1A	WATER	1480281.20	1556439.88	5474.16	CLEAR TO 60"
2A	GAS	1480268.91	1556450.55	5473.97	51"
3A	COMM	1480253.58	1556463.00	5473.79	CLEAR TO 72"
4A	COMM	1480250.54	1556460.80	5473.85	39"
5A	F.O.	1480234.35	1556477.98	5473.56	58"



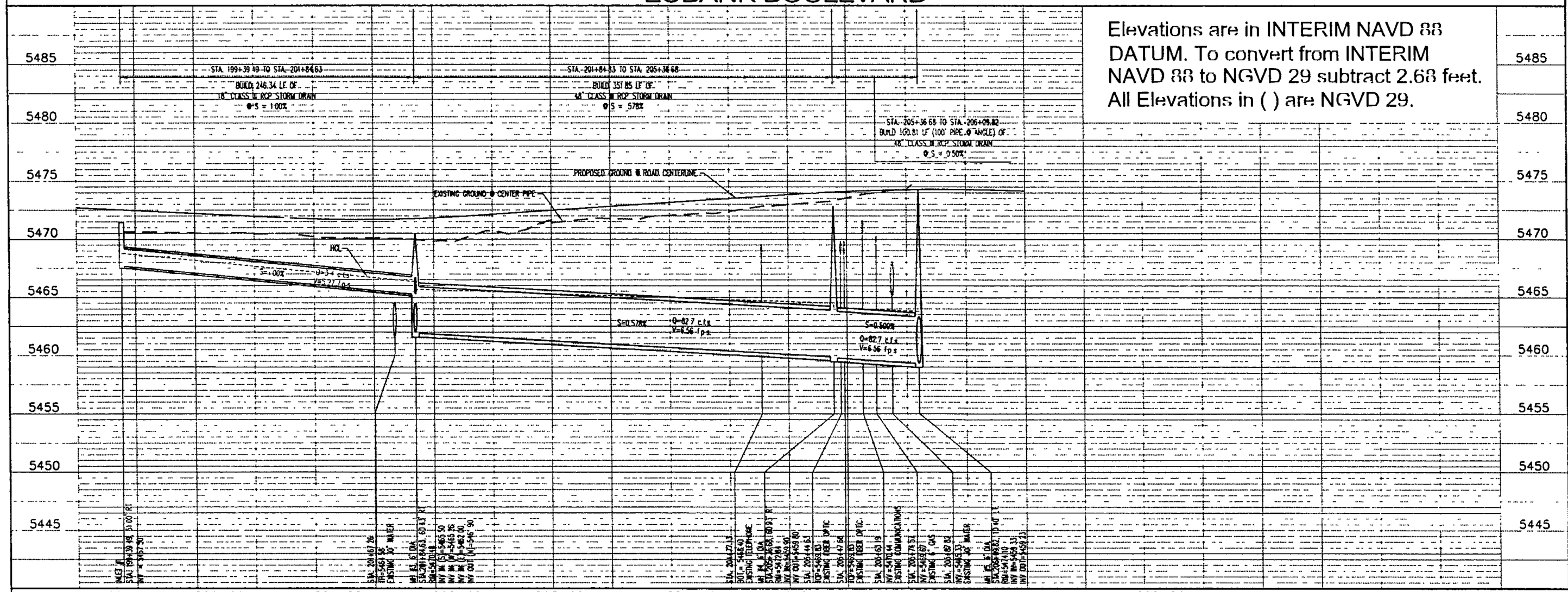
CONTROL POINT DATA TABLE (GROUND COORDINATES)			
POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956.90	1555960.12	5463.22
BHI CP 516	1478043.06	1555862.87	5461.77
BHI CP 517	1477917.89	1557166.71	5480.08

NOTES:
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM

50 25 0 50
1"=50'

STORM DRAIN TANGENT DATA

NUMBER	BEARING	DISTANCE
T1	N01°51'44"E	246.26'
T2	N00°31'33"E	351.85'
T3	N44°07'12"W	100.81'
T4	S66°38'37"W	118.07'
T5	N71°53'13"W	38.49'



Elevations are in INTERIM NAVD 88 DATUM. To convert from INTERIM NAVD 88 to NGVD 29 subtract 2.68 feet. All Elevations in ( ) are NGVD 29.

SCALE 1" = 50' (HORIZ)
1" = 5' (VERT)

P:\00268\Hydro\Design\PMP\00268pmp03-rev.dwg
Jun 19, 2002 - 4:23pm

AS-BUILT INFORMATION
CONTRACTOR — DATE
WORK STAKED BY — DATE
WORK INSPECTED BY — DATE
ACCEPTANCE BY — DATE
FIELD DIRECTION BY — DATE
DRAWINGS CORRECTED BY — DATE
MICRO-FILM INFORMATION
RECORDED BY — DATE
NO

BENCH MARKS
ACS Aluminum Cap stamped "6-1-21, 1986/92" Intersection of Central Ave & Elizabeth St. Geographic Position (NAD 1927), in feet N.M. State Plane Coordinates (Central Zone) X=417,975.01, Y=1,481,220.09 (Grid) Elevation=5500.67 NGVD29, in feet. Interim NAD 83 in feet [X= 1,558,774.59, Y= 1,481,809.99] (Ground) Ground-to-grid factor 0.99964527 Transformed to Interim NAVD 88 by Carpscon V5.11.08 [Elevation= 5503.35 Interim NAVD88, in feet]

SURVEY INFORMATION
FIELD NOTES
NO. | BY | DATE

ENGINEER'S SEAL

No.	Date	REMARKS	By
		REVISIONS	
		DESIGN	

Designed By M.P.C. DATE 04/02
Drawn By M.T.D. DATE: 04/02
Checked By: M.P.C./C.W.H. DATE: 04/02

Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

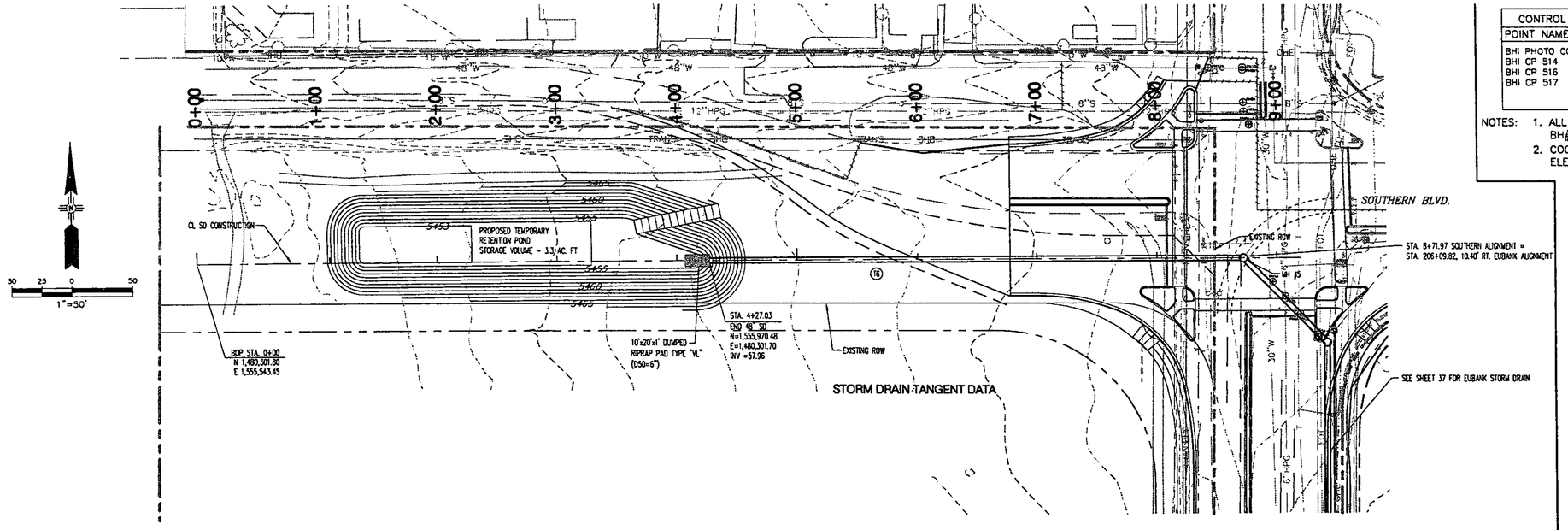
EUBANK BOULEVARD IMPROVEMENTS
STORM DRAIN PLAN & PROFILE - LA ENTRADA TO SOUTHERN
STA. 199+00 TO STA. 211+00

Design Review Committee	City Engineer Approval	Last Design Update	Mo./Day/Yr	Mo./Day/Yr

City Project No	Zone Map No.	Sheet	Of
6061.01	L-21&M-21	37	114

BHI JOB NO 00268

MARCH 2002



CONTROL POINT DATA TABLE (GROUND COORDINATES)			
POINT NAME	NORTHING	EASTING	ELEV
BHI PHOTO CONTROL △			
BHI CP 514	1479956.90	1555960.12	5463.22
BHI CP 516	1478043.06	1555862.87	5461.77
BHI CP 517	1477917.89	1557166.71	5480.08

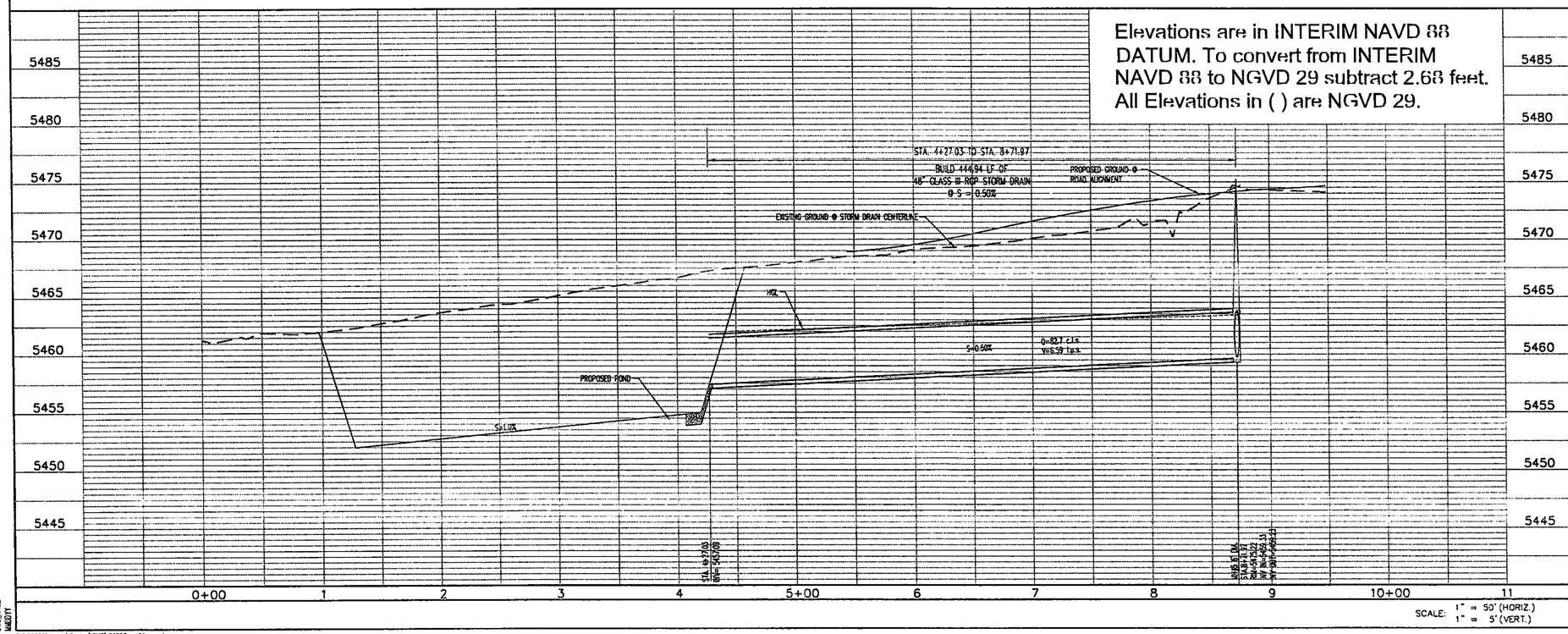
NOTES:
1. ALL CONTROL POINTS ARE 2" ALUM CAP STAMPED BH# ON 5/8" REBAR
2. COORDINATES ARE IN THE NAD83 DATUM AND ELEVATIONS ARE IN THE NAVD88 DATUM

STORM DRAIN PIPE TANGENT DATA

NUMBER	BEARING	DISTANCE
T6	N89°59'09"W	444.94'

SOUTHERN BOULEVARD

Elevations are in INTERIM NAVD 88 DATUM. To convert from INTERIM NAVD 88 to NGVD 29 subtract 2.68 feet. All Elevations in ( ) are NGVD 29.



SCALE: 1" = 50' (HORIZ.) 1" = 5' (VERT.)

AS-BUILT INFORMATION
CONTRACTOR DATE
NOTES REVIEWED BY DATE
STANDARDS ACCEPTANCE BY DATE
CERTIFICATION BY DATE
DRAWINGS REVIEWED BY DATE
MICRO-FILM INFORMATION
RECORDED BY DATE
NO.

BENCH MARKS
ACS Aluminum Cap stamped "6-L21, 1986/92" intersection of Central Ave & Elizabeth St. Geographic Position (NAD 1927), in feet. N.M. State Plane Coordinates (Central Zone) X=417,975.01, Y=1,481,220.09 (Grid) Elevation=5500.67 NGVD29, in feet. Interim NAD 83 in feet (X= 1,558,774.59, Y= 1,481,809.99) (Ground) Ground-to-grid factor 0.99964527 Transformed to Interim NAVD 88 by Corpscon V5.11.08 (Elevation= 5503.35 Interim NAVD88, in feet)

SURVEY INFORMATION
FIELD NOTES
NO. BY DATE

ENGINEER'S SEAL

No.	Date	REMARKS	By
		REVISIONS	
		DESIGN	

Designed By: M.P.C. DATE: 04/02
Drawn By: MTD DATE: 04/02
Checked By: M.P.C./C.W.H. DATE: 04/02

Bohannan Huston
Courtyard One 7500 JEFFERSON NE Albuquerque NEW MEXICO 87109
ENGINEERS PLANNERS PHOTOGRAMMETRISTS SURVEYORS SOFTWARE DEVELOPERS

CITY OF ALBUQUERQUE
PUBLIC WORKS DEPARTMENT

EUBANK BOULEVARD IMPROVEMENTS
DETENTION POND OUTFALL PLAN & PROFILE
STA. 0+00 TO STA. 9+00

Design Review Committee | City Engineer Approval | Last Design Update | Mo./Day/Yr. | Mo./Day/Yr.

City Project No. 6061.01 | Zone Map No. L-21&M-21 | Sheet 38 Of 114

BHI JOB NO. 00268 MARCH 2002

DWG_NAME
MMDDYY
P:\00268\Hydro\Design\PNP\00268prp04-rev.dwg
Jun 19, 2002 - 4:24pm

BOHAN HUDSON, INC

Courtyard One

7500 IEF [illegible]

Albuquerque

NEW MEXICO [illegible]

voice 505 823 [illegible]

505 798 [illegible]

www.bhnc.com